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POWER DIVISION

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Journal of the
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FURNAS HYDROELECTRIC PROJECT[a]

By James W. Libby,[1] M. ASCE

SYNOPSIS

This paper gives background data and describes the general design arrangement of Furnas Hydroelectric Project, now under construction on the Rio Grande in the state of Minas Gerais, Brazil. Ultimately Furnas will have an installed capacity of 1,200,000 kw, placing it among the world's largest hydroelectric developments.

INTRODUCTION

Brazil's population and industry is concentrated along 4,600 miles of Atlantic coastline. The states of Sao Paulo, Minas Gerais, Rio de Janeiro, and Espirito Santo have had, as late as 1954, about 45% (25 million) of the nation's population. Also located within these states was about 80% (only 2-1/4 million kw) of the entire installed electric generating capacity. The demand for additional power is urgent and increasing at a rate in excess of 10% compounded annually.

Thermal plant construction would appear to be an immediate method of overtaking the load demand, and private utilities have utilized such construction to a limited extent. However, as fuel must be imported, it substantially increases the cost of this type of energy. Nuclear energy with its present high cost is precluded as a solution to the immediate demand.

Brazilian officials have studied a number of major hydroelectric sites and have concluded that the early development of some of these sites presents the

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[a] Presented at the October, 1959 ASCE Convention in Washington, D. C.

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best answer to their power needs. One of the most desirable, located along the Furnas reach of the Rio Grande, is presently under construction.

Preliminary investigations of the Furnas site were initiated in 1954 at the request of Centrais Eletricas de Minas Gerais (CEMIG). When detailed field investigations began later in 1956, CEMIG invited Brazilian Traction's Sao Paulo Light and its technical unit (COBAST) to assist. As the power situation grew more critical, the Brazilian Federal Government formed a new organization, on September, 1957, known as Central Eletrica de Furnas (FURNAS). It is comprised of the Federal Government as a major sponsor, the states of Sao Paulo (DAEE) and Minas Gerais (CEMIG), Sao Paulo Light and Cia. Paulista de Forca e Luz. FURNAS has its engineering headquarters in Rio de Janeiro with a field office at the Furnas site.

REGIONAL POWER DEVELOPMENT

The market area and principal load centers are shown on Fig. 1. Recent estimates made by major utilities serving this area indicate the power requirements will be about 4,000,000 kw in 1960, 6,400,000 kw in 1965, and in excess of 10,000,000 kw by 1970.

This area had an installed capacity of 2,225,000 kw in 1954, and 2,500,000 kw at the end of 1957.

Major generating additions, other than the Furnas project, contemplated within the next several years are limited. The Peixoto plant downstream from Furnas is adding 90,000 kw. The Tres Marias project now under construction will have two units (130,000 kw) in operation by 1961 with two additional units to be added in the near future. The final installation will consist of eight units totalling 520,000 kw. One thermal installation is currently undergoing an expansion of 250,000 kw.

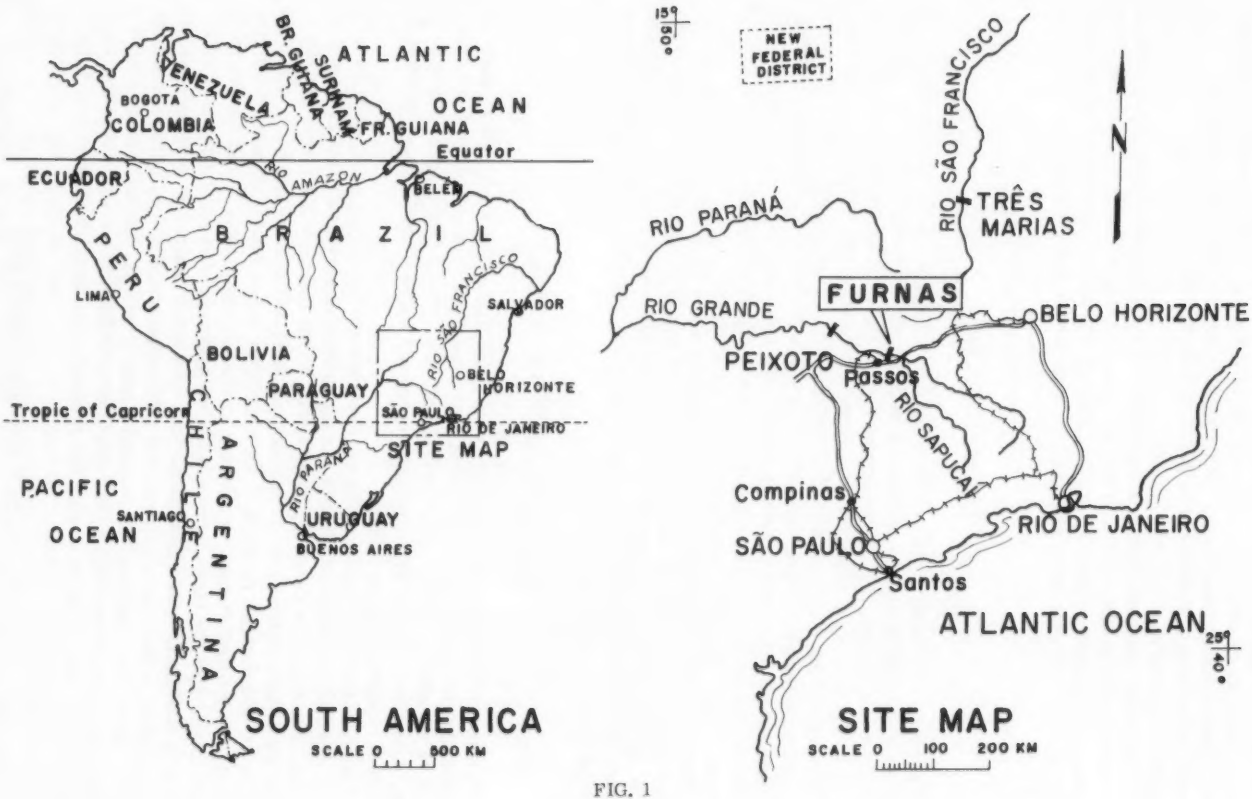
The region is faced with two other problems in its expansion program. One problem arises because service is supplied at both 60 cycles and 50 cycles, hampering interconnections. The other problem is the absence of a major transmission grid interconnecting the various power producing and distributing facilities. Little advantage can be taken of load diversity, surplus capacity, and energy between areas. Major additions to the transmission grid are planned coincident with the completion of the Furnas and Tres Marias projects.

SITE LOCATION

The Furnas site is located in the southwestern part of the State of Minas Gerais on the Rio Grande, a major tributary of the Parana (Fig. 1). It is about 320 km (200 miles) north of Sao Paulo, 280 km (175 miles) southwest of Belo Horizonte, and 400 km (250 miles) northwest of Rio de Janeiro.

Upstream from Furnas are two existing small developments with potential sites for two more on the main stem of the Rio Grande and another on a tributary. The aggregate installed capacity and usable storage of these are small in comparison to the Furnas project and will have an insignificant effect on Furnas operation.

Immediately downstream from the Furnas site is the existing Peixoto plant of Empresas Eletricas. After the completion of the Furnas project, the Peixoto plant will receive material benefits due to increased river regulation. Next downstream from Peixoto is the Estreito site which is under investigation by


ATLANTIC OCEAN
VENEZUELA
BOGOTA
COLOMBIA
BR. GUYANA
SURINAM
FR. GUIANA
Equator
ECUADOR
RIO AMAZON
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SITE MAP
PACIFIC OCEAN
SANTIAGO
CHILE
ARGENTINA
RIO PARANÁ
URUGUAY
BUENOS AIRES
SOUTH AMERICA
SCALE 0 500 KM
NEW FEDERAL DISTRICT
RIO SÃO FRANCISCO
TRÊS MARIAS
RIO PARANÁ
FURNAS
RIO GRANDE
BELO HORIZONTE
PEIXOTO
Passos
RIO SAPUCAY
Compinas
RIO DE JANEIRO
SÃO PAULO
Santos
ATLANTIC OCEAN
SITE MAP
SCALE 0 100 200 KM

FIG. 1

the Empresas Eletricas. This site when developed will probably have an installed capacity of about 800,000 kw. It will depend upon Furnas and Peixoto for regulation of stream flow. There are perhaps ten other potential sites downstream from the Estreito site which will benefit from regulation by the Furnas reservoir.

The closest railroad terminates about 32 km (20 miles) downstream from the Furnas site at the town of Passos. An existing state highway runs from Passos and passes within 8 km (5 miles) of the site. Improvement of both railroad and highway were required as well as the construction of an access road.

INVESTIGATIONS

Based on information obtained from early field work, preliminary office studies were performed to compare a number of alternate layouts at various sites. A single high dam at the lower end of the Furnas Canyon proved to be the most attractive. A report was released in September, 1955, which included recommendations for an expanded investigation of the site and reservoir area. The recommendations included surface and subsurface investigations, a drilling program, topographic surveying of dam and reservoir area, search for suitable construction materials and studies of alternate means of transport and access. This work was started almost immediately.

Meanwhile, the geologic and hydrologic data were reviewed, reservoir area-capacity curves redrawn from improved field data, and new power studies were made. During this period a number of consultants visited the site. Recognized authorities in geology, soil mechanics, heavy construction, hydroelectric power development, etc., were engaged.

SELECTION OF TYPE OF DAM

Considerable study was devoted to the selection of the best type of dam. It was determined early in the program that the differential cost between various types of structures would be relatively minor. Final selection of the type of dam, therefore, would be largely governed by subsurface findings and availability of construction materials.

The site itself was susceptible of development by either a fill-type dam or one of several concrete structures. Early indications favored a concrete gravity dam. Later, exploratory drilling revealed that the right abutment would require excessive excavation to prepare an adequate foundation for a concrete structure. Furthermore, much of the aggregate would have to be manufactured. Brazil has limited cement-producing facilities, and these would be severely taxed to produce the large quantities of cement required for a concrete structure and still meet other commitments. As more materials information was accumulated, it became apparent that a fill-type structure had distinct advantages. Also, the construction scheme would be simplified and would permit a faster construction schedule than would be possible with a concrete structure. The decision was made to design the project substantially as shown in Fig. 2.

HYDROLOGY AND METEOROLOGY

The drainage area of the Rio Grande upstream from the Furnas site is about 51,000 sq km (20,000 sq miles). The river rises in the Serra de Mantiqueira

in South Central Minas Gerais and flows generally westward to its confluence with the Parana.

A reliable gage has been maintained at Sao Jose da Barra, a short distance above the dam site, since 1930. By correlation with other stations, records were extended to mid-1928 in order to include 1929, which was a year with extremely high rainfall. Inflow between Sao Jose da Barra and the dam site is

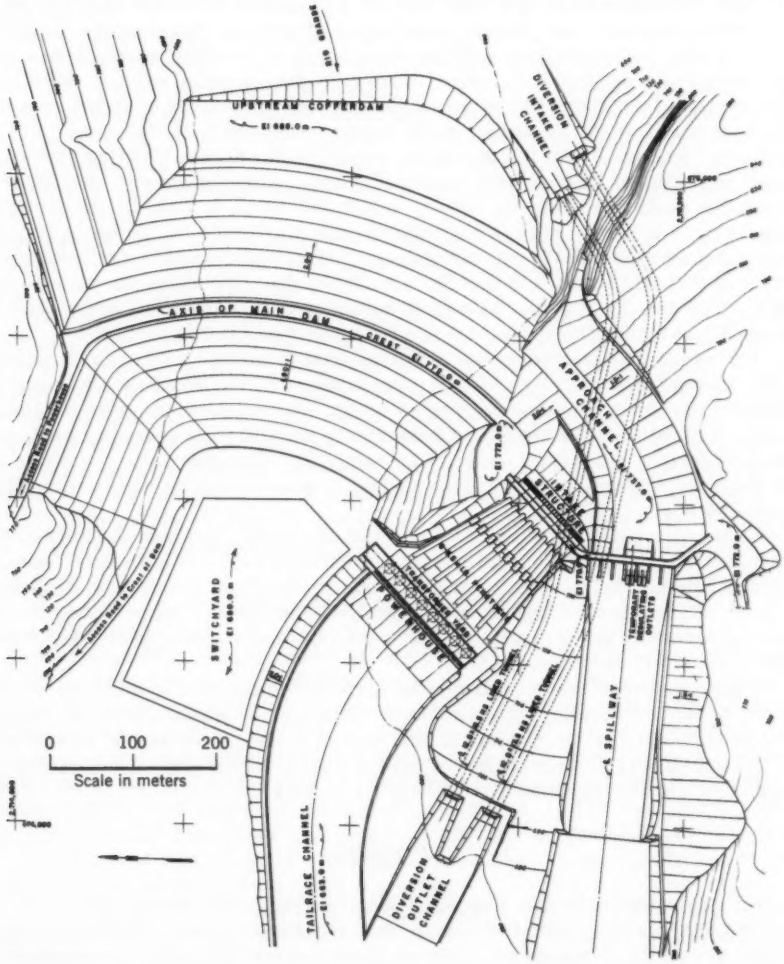

FIG. 2.—GENERAL PLAN OF FURNAS PROJECT

negligible, so the gage records were used for natural discharges at Furnas. Mean monthly flows have varied from a minimum of about 210 cms (7400 cfs) to a maximum of 4130 cms (146,000 cfs). The mean flow for the period of record is approximately 900 cms (31,700 cfs).

The flow record includes one extremely high year, and two protracted low periods. The first low period began about mid-1932 and extended into 1935. The second began in mid-1952 and lasted until early 1957. For purposes of power production, the latter was more critical because of its longer duration.

Flood potential is determined by the flow of tropical maritime air. The basin is mountainous, with maximum elevations reaching nearly 3,000 m (10,000 ft) and the headwaters are only 105 km (65 miles) from the Atlantic ocean. Circulation of air over Brazil is governed largely by a permanent high pressure area over the Atlantic with its center east of Uruquay, and by the position of the intertropical front. When it swings south, as it usually does in the January-March period, this front lies in a V-shaped pattern over the continent, with its southern tip extending to the northern boundary of Paraguay. Tropical maritime air from the northeast flows up the Amazon Valley toward the front and causes intense precipitation there. Simultaneously, maritime air from the east, also flowing toward the front, strikes most of the Brazilian coast. Air from the northern flow can enter the valley of the Sao Francisco when the intertropical front lies unusually far to the south. Ordinarily very little of this air will reach the Rio Grande basin. Air from the eastern flow will cross the Rio Grande whenever behavior of the front is normal, and the flow will become stronger as the front approaches the basin.

The basin is protected on the north and east from low level maritime air by 5,000-ft barriers. Except near the headwaters of the Sao Francisco and Rio Grande, the northern barrier is quite broad, while that on the east consists of a single ridge near the coast. Convectively unstable air from the east, carried by strong winds aloft, can cause heavy precipitation over the whole Rio Grande basin. However, the rain potential of these storms is lower than those from the north.

Although locally there are areas to the south that experience intense rainfall, general topographic configuration combined with the necessary easterly wind precludes the possibility of extended storms from the west or southwest. It is of interest to note that a careful study of the recent severe floods in Uruguay is currently being conducted with reference to storms approaching the Rio Grande basin from the south.

It was concluded that a spillway design flood would have to be produced by a storm from the north, with some lesser effect from a simultaneous easterly storm. The computations showed that such a storm would result in a 10,000-yr flood with a peak daily inflow of 18,000 cms (635,000 cfs), and a ninety-day volume of 945,000 cms-days (33,400,000 cfs-days).

GEOLOGY

Rock formations in the Furnas region belong to the Minas and Itacolomi series which were derived from sediments deposited in a Pre-Cambrian inland sea known as the Minas Sea. These sediments were subjected to considerable folding and shearing. Fine beach sands became sandstones, which later were fused by high temperatures. Finer sediments became indurated into shales, which later were metamorphosed into the sericite schists that are found interbedded with quartzite. Quartzites rich in mica represent sands that were mixed with silt or clay.

The Minas series is composed chiefly of sericite or chlorite phyllites, medium-grained, impure quartzites, magnesian limestone and itabirites. The

Itacolomi series includes great thicknesses of thin-bedded, fine-grained white, micaceous quartzites. They grade into the fine-grained laminated rocks rich in schistose elements that are known as Itacolomites.

First visits to the site were made in mid-1954. An exploratory drilling program was initiated in October, 1955, and was largely completed by mid-1958. An extensive grid of drill holes was logged, and geologic profiles and sections were prepared.

The left abutment at the Furnas site is composed of a finely grained highly jointed quartzite. The surface of suitable bedrock averages from 5 to 10 m in depth below natural ground surface over most of the left abutment except at the spillway area where maximum depth is about 35 m (115 ft). The general strike is North 25° West, with the dip 15° to 20° to the Southwest, or downstream and away from the river channel. Bedding planes form joints consisting of very thin layers of clay or fine silts which have been metamorphosed into micaceous planes.

In the main river channel there is a deep diagonal fault-formed trough. This has been intruded by a diabase dike that has gouge and fractured rock on both sides.

The right abutment is composed of thick beds of thinly laminated quartzites separated by relatively deep bands of micaceous schists that are fractured and friable near the surface. There has been considerable decomposition and weathering of the schists and a talus slope has formed at this abutment. Removal of the talus and weathered material will be necessary.

Areas containing decomposed schist are present at both abutments. It is expected these materials will be impervious, except where extreme weathering has taken place. Joints in the quartzites usually are tight and little grouting will be required in most areas. Water testing indicates little difficulty will be experienced with leakage.

MATERIALS INVESTIGATIONS

As investigations progressed it became apparent that a fill-type structure was better suited to site conditions than was a concrete structure. This resulted in an intensive search to locate sufficient quantities of the various types of material to be used in the dam. At the same time a testing program was instigated in which routine classification and strength tests were carried out to determine the characteristics of all such materials. Specialists in engineering geology and soil mechanics were consulted. Field tests have shown that most excavated material will be suitable for the random fill, and that rock excavation in the intake, spillway and powerhouse areas can be utilized for the rock shell portion of the dam.

Because of the slabby nature of the quartzite fragments to be used for the rock shells, it was considered advisable to make special investigations of the strength characteristics of this material. The U. S. Bureau of Reclamation in Denver was retained to perform large scale triaxial compression tests on the rockfill using high all-around pressures. The assumed strength parameters used in the preliminary stability studies would be reviewed and modified where necessary after the results of these tests became available.

Suitable earth core material is available on the right bank about a mile upstream from the dam axis and on the left bank about 4 miles from the site. Adequate transition zone material is also available. Routine tests of core and

transition zone materials have been performed in Brazilian laboratories and have determined the relative stability of the material in the various borrow pits. In addition, test strips have been constructed at the site to correlate field and laboratory compaction results.

Coarse aggregate for concrete will be manufactured from quarried material at areas within reasonable hauling distance from the project. Rock at the project site generally breaks into slabs and blocks due to its hard, brittle nature, and the presence of bedding planes and joints. It can be used to manufacture aggregate, but such aggregates would produce harsh concrete. Limited quantities of sand are available in the stream bed of the Furnas River and tributaries for concrete.

DAM AND RESERVOIR

The preliminary investigations and report considered a number of combinations for development of the Furnas reach of the Rio Grande. Economic studies justified the highest dam that could be considered. Maximum elevation was limited by low saddles in two rims in the upper reaches of the reservoir rather than by physical conditions at the site. Normal pool level was set at elevation 766 m, and at this elevation one saddle dam is required to prevent the reservoir from spilling over into the Sao Francisco River basin. The maximum gross head will be slightly less than 100 m (328 ft) with the exact value being dependent on the volume of tailrace excavation.

At the normal pool level, a lake will extend approximately 217 km (135 miles) up the Rio Grande and 190 km (120 miles) up the Rio Sapucai. Gross reservoir capacity will exceed 21 billion cu m (about 17 million acre-ft). Usable storage in a 16-m (52.5 ft) drawdown exceeds 14 billion cu m (about 11.5 million acre-ft). The storage is adequate to control completely the run-off of the Rio Grande better than 95% of the time.

In early planning, consideration was given to limiting the maximum level to El. 750 m. Economic considerations proved the inadvisability of restricting the water level to this lower elevation because Furnas would lose 17% of its gross head, and 75% of its gross storage. Since there are no major storage sites above Furnas, loss of the reservoir capacity could not be overcome by a series of low dams that would develop all of the head.

The land to be inundated by Furnas reservoir is largely undeveloped. There will be some major railroad and highway relocations, and a few small communities will have to be re-established. Also, some good farm land will be lost to future production.

POWER CONSIDERATION

Since there is a market for the power as soon as it can be made available, it was important to realize optimum development of the resource at the earliest possible date. Although an extensive transmission grid does not exist at present, it is reasonable to assume that Furnas project will be interconnected to all of the major projects and load centers within the Belo Horizonte - Sao Paulo - Rio de Janeiro triangle. It can likewise be assumed that Estreito will be built in the near future so that Peixoto and Estreito will be both hydraulically and electrically integrated with Furnas. Existing and possible future upstream projects will have only minor effect on the power supply and negligible

effect on reservoir operation as compared with Furnas and can be ignored in any power study for the Furnas site.

Power potential of Furnas was determined by making a number of studies for the 29-yr period of record. First computations were performed manually, but final studies were accomplished with an electronic computer. The minimum system which was considered included Peixoto and Estreito. Studies also were made assuming auxiliary sources of power in addition to the above projects. As might be expected for a project with a reservoir having a high degree of stream flow control, it was found that Furnas could economically utilize almost all of the flow.

Annual load factors of the existing utilities are in the range of 58 to 65%. Computer studies were based upon load factors of both 0.50 and 0.60; the former to consider possible future conditions, the latter to represent more closely the existing load. It was found that the project could economically generate an average of about 5.4 billion kwh annually and would waste less than 1% of the water. The installed capacity would be limited to about 1,200 mw, which is adequate to serve the project's share of the average annual system load at system annual load factor. It is recognized that a project of this magnitude should be considered for possible future peaking operations at a relatively low load factor.

The owners decided, after due consideration, that funds required now for provisions for possible future capacity expansion would be more effectively utilized for additional energy producing plants. No provision, therefore, has been made for more than eight units at this project.

PRIME MOVERS AND GENERATING EQUIPMENT

Consideration of plant generation output and costs of various sizes of units led to adoption of a plant containing eight units, each capable of developing 150 mw, giving a total plant capacity of 1,200 mw. Present plans call for the first unit to be in service in mid-1962, with units two through four added at three-month intervals. No dates have been fixed for installation of the last four units, but it can be expected that they will be placed in service shortly following the initial installation.

Turbines were specified to deliver generator nameplate at near best gate at heads expected to prevail a large percentage of the time. Since the reservoir has a relatively small total drawdown which is utilized only in the driest years, the weighted average head is very high. This condition satisfied the Furnas Company's requirement that turbines be selected so they could not overload the generators by any appreciable degree (that is, not more than 6% to 8% at highest head). There results from this selection a small deficiency in capability under minimum head conditions. Since such a condition occurs infrequently (twice in the period of record) and then for only short intervals, this limitation was not considered serious by the owner. The turbines will have a nominal expected output of 192,500 metric horsepower (190,000 hp) when operating at a net head of 94 m (308 ft) at the point of best efficiency.

POWERHOUSE AND SWITCHYARD

The powerhouse shown in Figs. 2 and 3 will have an indoor type arrangement. Rainfall in Brazil during the wet season frequently exceeds 10 in. per

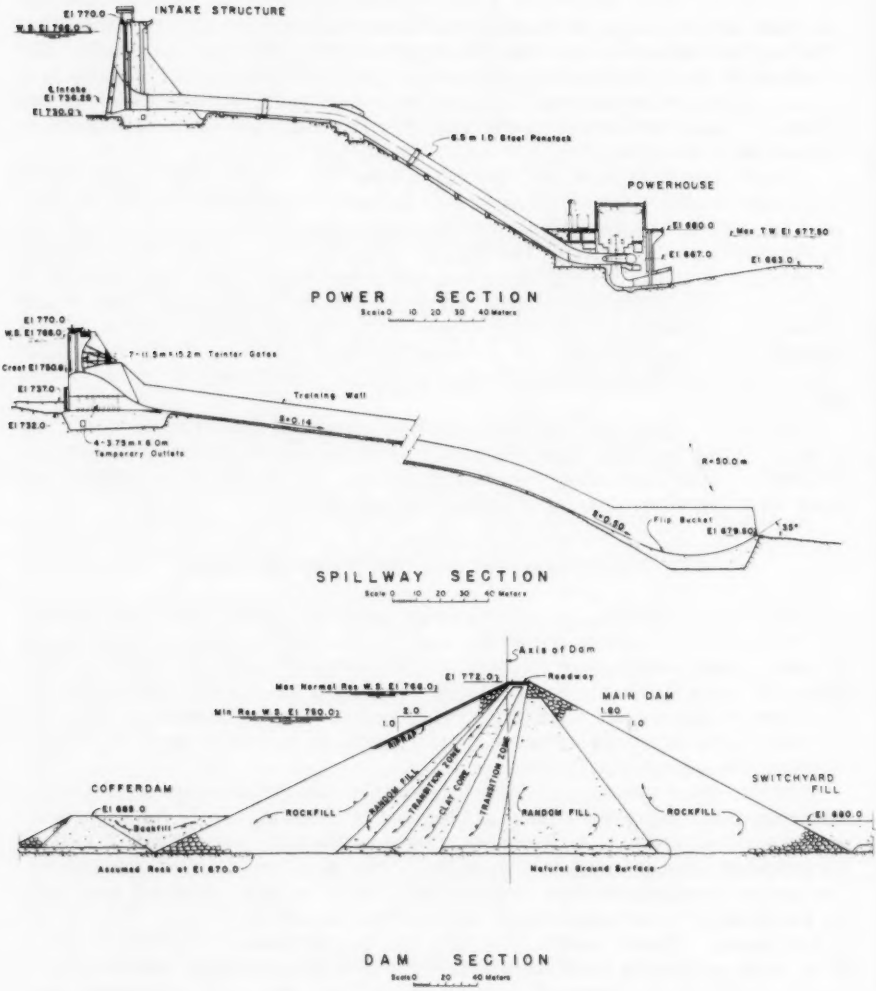

FIG. 3.—FURNAS PROJECT SECTIONS

month for a period of five or six months. If an outdoor type plant were used, it would be necessary to provide shelter for dismantling and maintenance of the units, untanking of transformers, etc.

Tailrace deck level is set well above maximum tailwater, with the generator floor depressed to reduce height of building and length of generator shafts. The transformer deck will be on the upstream side of the powerhouse. This arrangement simplifies the structural problems associated with support of these pieces of heavy equipment and fits well with the switchyard arrangement.

Each turbine, as shown in Fig. 3, will be served by a separate penstock 6.5 m (21.3 ft) in diameter. Adequate load and speed regulation will be possible without surge tanks. It is expected that an acceptable amount of extra $W R^2$ will be provided in the generators.

It was decided to use single phase transformers, a bank of three to each generator. Each transformer will be of the forced oil water cooled type and rated at 53,333 kva. Bank connections will be delta to grounded wye with line to line voltages 15 kv and 345 kv respectively. Physical size and simplicity of replacement and transport limitations were basic considerations in this selection. The units will be tied to 138 and 345 kv outgoing lines through a multiple bus switchyard arrangement.

Orientation of the powerhouse and intake structures was studied at considerable length. The dip, strike and cleavage pattern of the rock foundation required these structures to be located with their long axes parallel to the river channel for maximum economy and safety against sliding. However, such an arrangement would result in undesirable flow conditions to the intake. The arrangement selected, as shown in Fig. 2, was a compromise of all the factors involved.

To make maximum use of available head between the Furnas site and Peixoto reservoir, an economic study was performed to determine the extent of tailrace excavation. It was found that a major amount of excavation could be justified.

The switchyard will be located on Ilha do Sapo, which is across the tailrace from the powerhouse between two natural river channels. Material from required excavation will be utilized to raise and enlarge the Ilha to its required size and elevation.

INTAKE AND SPILLWAY

The limited space available and configuration of the site were basic considerations in the arrangement of the intake and spillway. When foundation and materials investigations supported selection of a fill type dam, the locations for the intake, spillway, and powerhouse were automatically fixed. That is, they should go on the left side of the channel. Conditions for this layout are not ideal; neither are they seriously deficient. Foundation materials on the left were far better than those at the right abutment, but as noted above, dip and strike of the rock somewhat limited powerhouse orientation. Sound rock elevation is fairly low in a relatively small area, but it is adequate for spillway and intake structures.

The spillway occupies the extreme left section of the site. In addition to the aforementioned gated weir, it will have a concrete-lined chute with a flip bucket at the lower end. Water will be discharged into the natural river channel well downstream from the powerhouse. A short concrete non-overflow section will connect the right end of the spillway to the powerhouse intake structure.

With the arrangement as planned, flow conditions other than ideal were to be expected. These were studied in a 1:100 scale model of the intake and spillway area for a complete range of reservoir elevations. As a result of the model studies which were performed in Brazil, certain modifications to the original design proved necessary.

The spillway design flood was developed from the known hydrologic and meteorologic condition discussed briefly earlier in this paper. It was concluded that the flood would have a peak inflow of 18,000 cms (635,000 cfs). Volume being of prime consideration, this flood would have 128,960 cms-days in the first 31 days, 231,280 cms-days in the next 28 days, and 183,520 cms-days in the last 31 days. Flood routing studies were performed using different assumptions and it was found that discharges less than 10,000 cms (353,000 cfs) would pass with no reservoir surcharge. This was true even though the reservoir was assumed to be at full elevation 766.0 m when the flood began.

The spillway will be designed to discharge 1300 cms (460,000 cfs) with the reservoir surcharged 3 m to El. 769.0 m. Discharges will be controlled by seven radial crest gates each 11.5 m (38 ft) wide by 15.2 m (50 ft) high. The gates will be provided with individual hoists. In addition, the intake gantry crane will be able to handle spillway stop logs and heavy lifts during maintenance operations.

The intake and spillway decks were set at El. 770.0 m to provide an extra meter of freeboard should the reservoir ever be surcharged to El. 679.0 m. The crest of the rockfill dam was fixed at El. 772.0 m to provide additional safety against overtopping.

The intake structure is designed as a concrete gravity dam with one monolithic block per penstock. An extensive drainage system of down holes with a drainage tunnel will be provided in the quartzite foundation to prevent excessive seepage pressures developing downstream of the intake and spillway crest structure. There will be conventional trash racks, with provision for raking and trash removal to be installed later if required. Gates will be individually cable hoist operated. A separate gantry crane will be provided for heavy lifts and raking operations. The right end of the intake structure will abut the quartzite cut slope at the left end of the main dam. Both intake structure and spillway decks will be wide enough for vehicular traffic.

It is expected that little grouting will be required under the spillway, nonoverflow and intake structure concrete. Water tests in exploratory drill holes have shown the quartzite to be relatively tight. This has been verified further by conditions encountered during excavation of the diversion tunnels.

DAM

The canyon at the Furnas site is 350 m (1,150 ft) wide on the axis at river level (Fig. 4). Natural ground extends well above maximum reservoir level on both abutments. Opposed to these favorable conditions the foundation in the river bed is traversed by a deep fault that has been closed with a diabase intrusion.

The section of the dam as shown in Fig. 3 was adopted after both circular arc and sliding wedge analyses showed that it was a structurally stable design. A machine language computer program was used to aid these studies. Strength parameters for all materials were obtained from laboratory tests performed in Brazil.

The core and transition zones are arranged as a compromise between an inclined core and a central vertical core. This arrangement was found to utilize available materials most effectively. The overburden material from the spillway excavation was found to be suitable for use in the transition zone to prevent particle migration from the core. Rockfill will be placed in layers and compacted by crawler-type equipment.

Horizontal curvature of the main embankment has been accentuated to improve abutment contacts and minimize the possibility of cracks developing in the core due to downstream movement. Because of the steepness of the left abutment (Fig. 5), special precautions will be taken to ensure that detrimental effects of differential settlement will not endanger the safety of the core. The upstream and downstream transition zones will be extended along the rock face.

FIG. 4.—SITE OF FURNAS PROJECT

Maximum height of the dam above foundation excavation will be about 120 m (395 ft). Exact elevation of deepest excavation will not be known until the main channel is unwatered. However, core drilling through the diabase dike indicates the material is sound, with the possible exception of fractured and decomposed material immediately adjacent to the diabase. Unsuitable material will be removed and replaced with impervious fill. Limited grouting is planned and will consist principally of test holes to check watertightness and grout take. Excavation to suitable foundation on the left abutment is expected to be nominal. The right side will require considerably more excavation to provide a suitable contact area for the clay core against sound impervious abutment rock.

Total embankment material is expected to be about 9,000,000 cu m (roughly 12,000,000 cu yd). About 600,000 cu m (800,000 cu yd) will be clay material

for the core and cutoff. Ample borrow area for this material is located on the left bank about 1 mile upstream of the dam. Transition zones will require 1,350,000 cu m (1,640,000 cu yd), sound rock 4,300,000 cu m (5,400,000 cu yd) and random fill about 2,275,000 cu m (3,000,000 cu yd). Required excavation will provide all main dam embankment materials except those necessary for the clay core. Materials from excavation unsuitable for the main dam will become a part of the switchyard fill on Ilha do Sapo or will be wasted in the river channel at the right of the island.

STATUS OF PROJECT

The major construction contract was awarded in mid-1958 to a joint venture. Work was started on the two 12.8 m x 13.5 m (42 ft x 44 ft) horseshoe-shaped

FIG. 5.—LEFT ABUTMENT, VICINITY OF DIVERSION TUNNEL INTAKE PORTALS.

diversion tunnels, driving by the heading and bench method. The bench was "holed through" in the latter part of June, 1959. Rock is generally excellent, though in a limited reach near the upstream portals schistose material was encountered, requiring support. At present the arch is being lined by use of the "Aliva" process, a European equivalent to shotcrete, with mesh reinforcing used as required.

Cofferdams confining the river into the deep central channel have been built on both sides of the river, with considerable excavation and scaling at each abutment. Present plans call for diversion through a channel on the right side during the next season, with the tunnels carrying only a portion of the diversion flow. Later, when the channel is closed and fill is placed on the right side, all flow will be diverted through the tunnels. Tunnels will be plugged after closure bulkheads are lowered and the reservoir starts to fill.

Due to its remote location, it was necessary to build housing and allied facilities at the site. This work, along with mobilization of men and equipment, required an extensive period of time and was partially completed by the owner prior to award of the construction contract. The owner will award separate supply contracts for furnishing all major items such as electrical and mechanical equipment, gates, cranes, hoists, special steel items such as penstocks and trash racks, gates, switchyard equipment, etc. These items will be furnished where possible by Brazilian manufacturers. Major items not available in Brazil will be imported.

COST AND FINANCING

Total project construction cost with four units installed, exclusive of transmission lines, is currently estimated to be $129,000,000 with an additional $40,000,000 required to bring the plant to its ultimate capacity. The Brazilian government, as principal sponsor, is furnishing the bulk of local currency requirements. It also has obtained a $73,000,000 loan from the World Bank for purchase of foreign materials and equipment. Additional funds in foreign currency will be required in order to install the four future units. No problems are anticipated in this regard, because Furnas has been shown to be an excellent project.

SUMMARY

Brazil, a country larger in land area than the continental United States and populated by 65 million persons, is faced with a severe power shortage. Furnas project on the Rio Grande in Southwestern Minas Gerais is one element in a growing program that will help to alleviate the lack of adequate electric power and energy. This project has a reservoir about 240 km (150 miles) long with over 17 million acre-ft of water, of which about 12 million acre-ft will be usable with a drawdown of only 17% of the gross head.

The initial installation will consist of four 150 mw units, with the ultimate capacity of eight units totaling 1,200 mw. The average annual generation with eight units will be about 5.4 billion kwh, all of which will be usable in the area load as soon as it can be made available. Present plans contemplate first unit on the line by mid-1962 with three additional units scheduled at three-month intervals. No firm schedule has been set for the last four units, but they will undoubtedly be required at an early date.

The main dam will be a composite fill-type structure containing approximately 9,100,000 cu m (12,000,000 cu yd) of material. It will be some 580 m (1,900 ft) long at crest elevation and will have a maximum height from deepest excavation to crest level of about 120 m (400 ft). A spillway and chute on the left abutment will pass nearly 10,000 cms (350,000 cfs) at normal full pool level and 13,000 cms (460,000 cfs) with a 3-m (10-ft) surcharge.

Although there is only one existing downstream project, potentially there are sites for about a dozen more. The firming effect of Furnas reservoir on these future projects will enhance its regional value considerably. Furnas would be considered a large project in any country. For Brazil it represents a tremendous undertaking, and it is a tribute to the courage and efforts of her people. Although they are utilizing foreign consultants, the bulk of the design work is being performed by Brazilians.

PERSONNEL

Central Eletrica de Furnas is headed by John R. Cotrim, M. ASCE, president, Flavio H. Lyra, technical director, B. Dutra, financial director and F. Von Ranke, chief engineer. Cia. Internacional de Engenharia e Construcoes, a Brazilian company associated with International Engineering Company, Inc., is preparing the construction drawings and specifications. The technical staff of Sao Paulo Light is available for consulting service. Other consultants include A. Casagrande and S. D. Wilson for soils; Portland P. Fox, F. ASCE, geologist; E. P. Eardley for transmission facilities; R. L. Hearn of Ontario, Hydro. Geological data given in this paper are based on the work of J. Cabrera, staff geologist of Sao Paulo Light. Meroz and McLellan of Newcastle upon Tyne, England, and Ebasco Services, Inc., were engaged by the client on related problems. Hydraulic model work was performed by the Escritorio Saturnino de Brito Hydraulic Laboratory in Rio de Janeiro.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

PIRATININGA STEAM-ELECTRIC GENERATING STATION

By O. L. Hooper,[1] F. ASCE and H. M. Estes[2]

SYNOPSIS

This paper describes the construction of the Piratininga steam-electric generating station and the hydroelectric system of which it forms a part. Particular attention is given to the problems of transportation unique to the location.

INTRODUCTION

The Piratininga steam-electric station and hydroelectric generating stations form the power system of the electric utility serving the most populous part of the state of São Paulo in Brazil. The hydroelectric system of the São Paulo Light S.A. - Servicos de Eletricidade is unique in the world since it utilizes, in an ingenious way, the unusual topography of the region.

The predominant feature of topography is the steep escarpment called Serra do Mar, varying in altitude from 900 m to 1,500 m, or 3,000 ft to 5,000 ft, which is near the Atlantic coast and forms the divide between drainage directly to the coast and drainage to the Rio Tiete flowing westward through Sao Paulo on its way to the Parana, which flows south to enter the Atlantic near Buenos Aires in the Argentine. The program of hydroelectric development has been based on diverting water from the Tiete and its westward flowing tributaries by gravity or by pumping to the large Billings Reservoir near the Serra, and on de-

Note.—Discussion open until September 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. PO 2, April, 1960.

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veloping approximately 2,350 ft of head in the sharp fall to sea level south of the Serra.

Fig. 1 shows the name and location of the streams, reservoirs, and plants that comprise the present hydroelectric system along with a diagrammatic profile of the diversion scheme. Edgard de Souza, Rasgão, and Porto Góis were low head, run-of-stream pondage plants on the Tiete which were constructed before and while the diversion over the Serra was being developed. Edgard de Souza plant has had the dam raised and pumps installed after constructing Pirapora Dam to start the waters of the Juqueri River and other tributaries of the Tiete toward the Billings Reservoir at the Serra.

The large Sorocaba Reservoir supplies regulated flow to the 56,124 kw Itupararanga plant and has no hydraulic relationship to other parts of the hydroelectric system. The Guarapiranga Reservoir was constructed at an early date to furnish potable water to the city of São Paulo, afford flood control on the Pinheiros River near the city and furnish some regulated flow to the Edgard de Souza plant on the Tiete.

The early concepts of the diversion scheme have been ably described by Adolph J. Ackerman, F. ASCE, in his biography of A. W. K. Billings.[3] The project now includes the Billings Reservoir with usable storage in it equivalent to 2 billion kw hr which discharges through the Summit control structure into Pedras Reservoir, which has adequate storage for weekly pondage. The first Cubatão plant at the foot of the Serra is supplied from Pedras through two tunnels to the valve house and thence through eight penstocks. The gross head is 720 m, or 2,360 ft, and the aggregate capacity of the horizontal shaft impulse units in the plant is 474,004 kw. The second Cubatão plant is an underground station near the first plant supplied from Pedras by a sloping tunnel. It now contains four 65,000 kw vertical shaft, and four-jet impulse turbines, and two similar units are being installed which will result in an aggregate capacity of 390,000 kw.

Initially, the water flowing to Cubatão plant consisted of the yield of the Pedras River plus the yield of the Grande River at the dam creating Billings Reservoir, which together average about 16 cu m per sec. After the Pedreira pumping plant and the Traicão Dam were installed, the releases from Guarapiranga Reservoir, averaging about 10 cu m per sec, could be pumped into Billings Reservoir. The Pinheiros River, which had been subject to serious floods that overflowed marginal lands, was straightened and then enlarged to accommodate the increasing diversions from the Tiete toward the Pedreira pumping plant. The Retiro intake structure, at the point where the Pinheiros joins the Tiete, controls the passage of water from the Tiete into the Pinheiros and limits the flooding of the Pinheiros by backwater from the Tiete. About midway along the Pinheiros canal, the Traicão pumping plant was installed. The flow of the Tiete upstream of Pirapora Dam that can be diverted to the Cubatão plants by pumping at Traicão and Pedreira averages about 60 cu m per sec. Billings Reservoir is so large with its 1.2 billion cu m, or 970,000 acre-ft, of usable storage, that regulation during the driest period of record extends for more than three years from full to empty and the firm hydro energy during the driest period is about 90% of the long-term average hydro energy.

It is worth noting that the small pumping heads on the diversion scheme, 19 m at Edgard de Souza, 5 m at Traicão and 26 m at Pedreira, compare to the

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3 "Billings and Water Power in Brazil," A Short Biography of Asa White Kenney Billings, by Adolph J. Ackerman, ASCE, 1953.

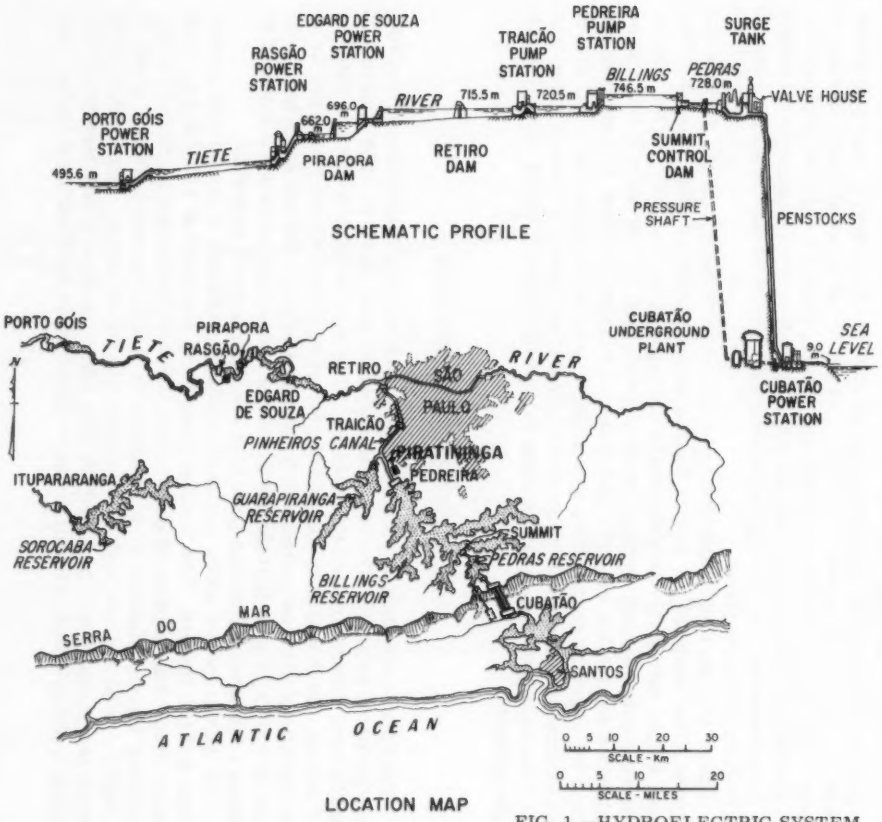


FIG. 1.—HYDROELECTRIC SYSTEM

SYSTEM GENERATING CAPACITY

HYDROELECTRIC	
CUBATÃO I - ORIGINAL	474,004 KW
CUBATÃO II - UNDERGROUND - PRESENT	260,000 KW
- ADDITION	130,000 KW
ITUPARARANGA	56,124 KW
RASGÃO	18,539 KW
PORTO GÓIS	10,905 KW
ISABEL	3,200 KW
SALESÓPOLIS	2,181 KW
QUILOMBO	1,818 KW
SMALL INSTALLATIONS	2,256 KW
SUB-TOTAL HYDRO PRESENT	829,027 KW
AFTER CUBATÃO ADDITION	959,027 KW
DIESEL	
SÃO SEBASTIÃO	649 KW
STEAM	
PIRATININGA - PRESENT	160,000 KW
- ADDITION	250,000 KW
SUB-TOTAL STEAM	410,000 KW
SYSTEM CAPACITY	
TOTAL - PRESENT	989,676 KW
- AFTER ADDITIONS	1,369,676 KW

net effective head of about 690 m at the Cubatão plants. Both the Traicão and Pedreira plants have reversible pump-turbine units so that during peak load hours consumption of energy for pumping can cease and the units can supply capacity and energy to the system.

In this situation, the steam-electric plant at Piratininga has increased the firm system energy supply as well as the generating capability, and permits greater utilization of the hydro energy available in better-than-dry years. In addition, the location of the steam plant near the major load center has improved the electrical characteristics of the transmission and subtransmission system. The system of the São Paulo operating company operates at 60 cycles and is interconnected through frequency changers with the 50 cycle system of the Rio de Janeiro operating company, both companies being subsidiaries of the Canadian Corporation, Brazilian Traction Light & Power Company, Ltd.

LOCATION AND SITE

The Piratininga steam-electric station which is shown on Fig. 2 is located adjacent to the Pedreira pumping station. This location was selected for ease of integrating it into the system, it being on the Pinheiros canal which serves as part of the hydraulic system discussed earlier. The canal is highly contaminated by sewage, but serves as a tolerable source of the condenser-circulating water. The site is largely of a swampy nature, having a layer of organic clay as the topsoil. Beneath this are varying depths of granite residue in all stages of decomposition. A sound granite bedrock is found at depths varying from 0 to 50 ft below the surface.

There is a knoll of sound granite which reached nearly to the surface and proved to be adequate for supporting the major portion of units 1 and 2. This knoll helped define the most desirable location for the station. As the station was extended the rock surface became lower, and for the units 3 and 4 considerable excavation of soft material was required so that column footings could be placed on rock. This soft material was replaced by sand and gravel fill to provide support for floors and auxiliary equipment. Ground water was not a serious problem in performing this work, it being easily controlled by open pumping.

OPERATING CHARACTERISTICS

The Piratininga plant is oil burning and uses residual fuel oil from Brazilian and imported crudes. The initial installation completed in 1954 consists of two turbine generators and two boilers. The turbine generators are rated at 80,000 kw. Each boiler is capable of delivering 850,000 lb of steam per hr to the throttle of the associated turbine at a pressure of 850 psi and a temperature of 925° F. The relatively low pressure and temperature were selected to obtain a low cost station for peak load and simplicity of operation.

The extension now under way involves the installation of two turbine generators and two boilers. The turbine generators are rated at 125,000 kw. Each boiler is rated as capable of delivering 1,000,000 lb of steam per hr to the associated turbine throttle at a pressure of 1,800 psi and a temperature of 1,000° F. The boilers and turbines are the reheat type. After reheat, the steam temperature will be 1,000° F. The higher pressure, temperatures, and reheat were selected to obtain greater efficiency in anticipation of higher load

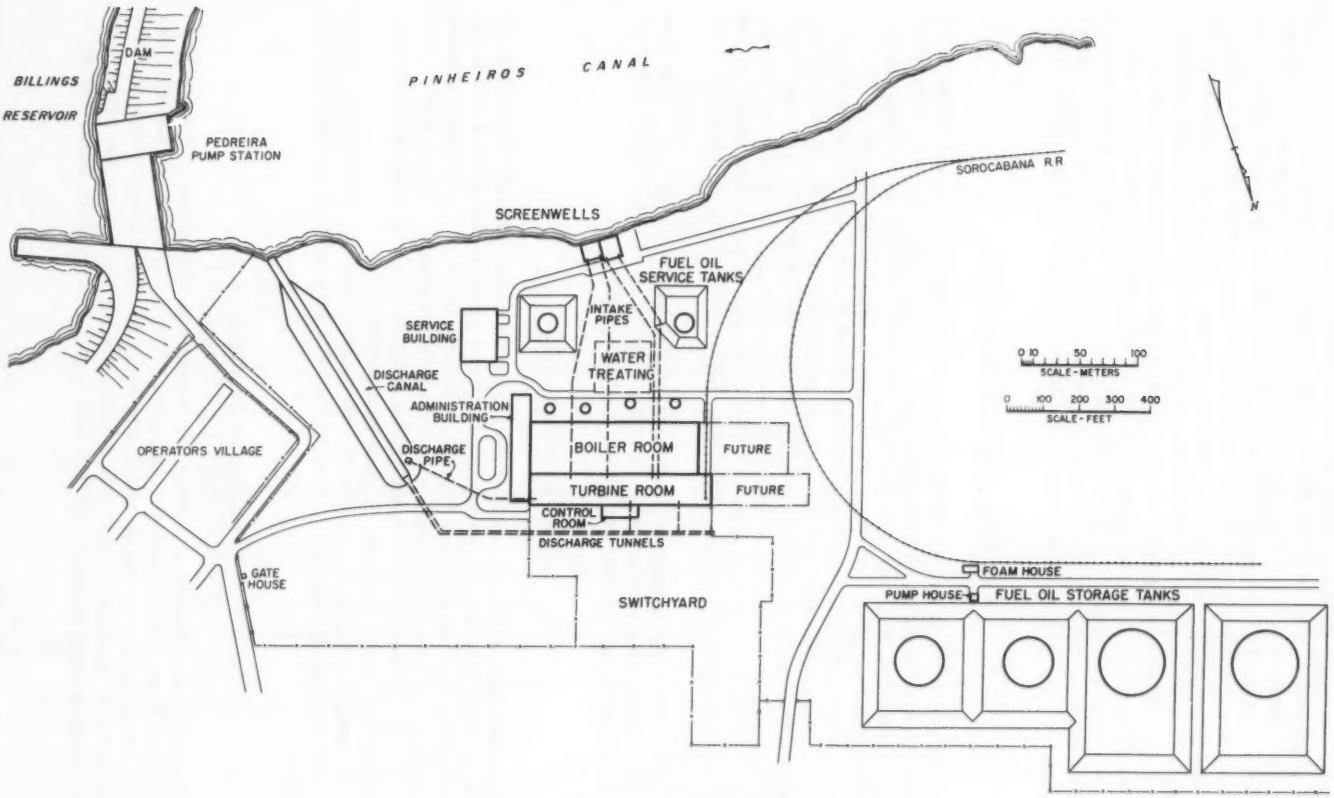


FIG. 2.—PLOT PLAN FOR PIRATININGA

factors with high cost fuel. The first of these units is scheduled to start commercial operation in March 1960 and the second four months later.

The two existing generators are rated at 13,800 v, 100,000 kva, 0.80 pf, and step-up transformers are provided to deliver current to the system at 88 kv. The two new generators are rated at 14,400 v, 147,058 kva, and 0.85 pf, and step-up transformers are provided to deliver current to the system at 230 kv. Cooling water is taken from the canal by vertical shaft pumps located in the canal bank and the water is later returned to the canal. At full load, approximately 74,000 gal of water per min are used by each of the two-pass condensers for the 80,000 kw units and approximately 61,000 gpm will be used by each of the two-pass condensers for the 125,000 kw units, making a plant total of 270,000 gpm or 600 cfs, or 17 cu m per sec. The reduced quantity of water required for the larger units is due to the higher efficiency of the reheat units.

MATERIALS OF CONSTRUCTION

Under pressure to use as limited an amount of materials imported from the United States and as little structural steel as practicable for the Piratininga plant, reinforced concrete was used for the major portion of the structures. Referring to the cross section on Fig. 3, it will be observed that all of the turbine room has been made of reinforced concrete except the roof trusses, purlins and minor walkways and that a high percentage of the boiler-supporting structure was made of concrete with structural steel used only for those parts directly supporting the boiler where connections to concrete would be complicated, and for the light platforms. This selection of materials satisfied the restrictions imposed for import licenses and the availability of materials from local sources.

Walls are of brick masonry covered on the outside with stucco. This type of construction is consistent with Brazilian practices where adequate low cost labor skilled in this type of construction is available.

FOUNDATIONS

Foundations for the power plant building are reinforced concrete carried to bedrock. Although some deep foundations were required for units 3 and 4, the use of piling was not believed feasible due to the depth of decomposed granite intermixed with granite boulders overlying the bedrock. It was feared that piles driven in this material might prove inadequate where stopped by the numerous large boulders in the decomposed material.

The auxiliary buildings are founded on compacted fill used to replace clays and soft decomposed granite. Similar fill was also used as the base for the oil tanks.

CONCRETE DESIGN

There is considerable conflict in the conception of concrete design between the United States and Brazilian engineers. The Brazilians, in general, use higher stresses than would be allowed by the standards of the American Concrete Institute (ACI) particularly in column design. For the design of the concrete work in units 1 and 2, the ACI code was followed throughout, except in

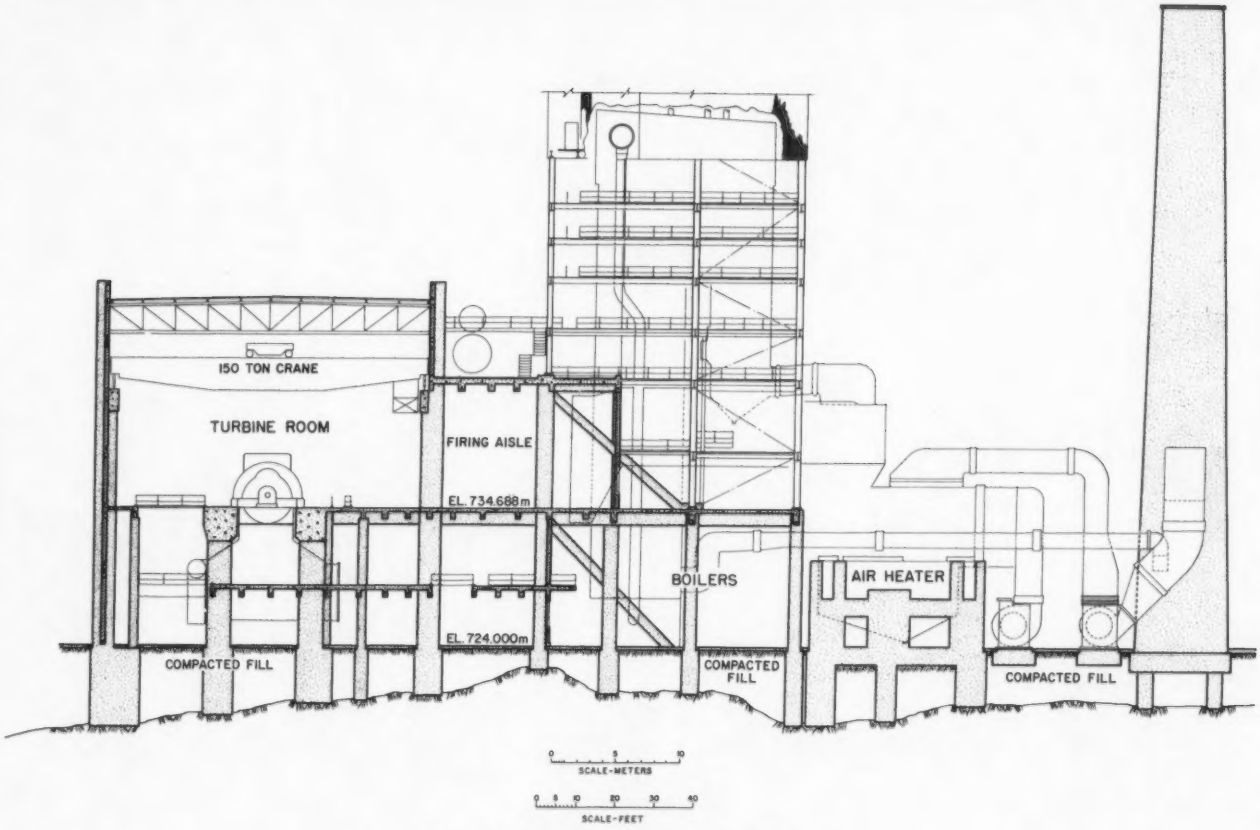


FIG. 3.—CROSS SECTION OF UNITS 3 AND 4, PIRATININGA

cases such as bonding of plain bars, the only type available, where the Brazilian code was more restrictive. This resulted in the size of columns appearing to be disproportionately large, and the low bond stress used created difficulties with the placing of concrete and reinforcement due to the numerous small bars required. For units 3 and 4, the design stresses were re-evaluated and compromise stresses for columns and United States allowances for bond were used, as shown in Table 1. The use of these stresses resulted in a rational design with an adequate factor of safety under the climatic conditions involved.

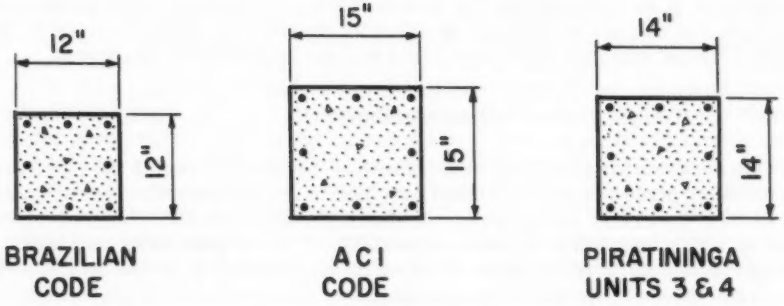
TABLE 1.—COMPARISON OF ALLOWABLE UNIT STRESSES FOR CONCRETE DESIGN 3,000 LB CONCRETE

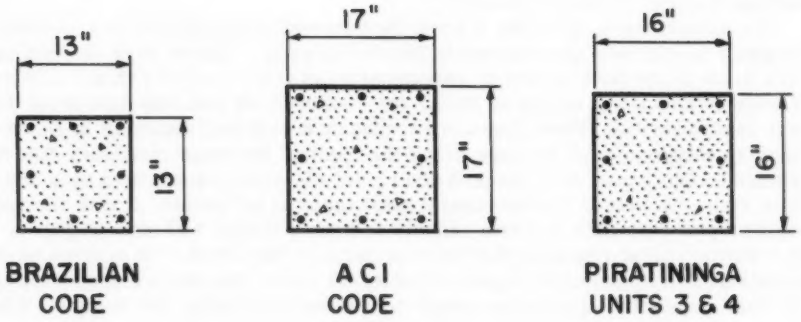
Item	Brazilian Code, in psi	ACI 1956, in psi	Used for Units 3 and 4 psi
Extreme Fiber Stress			
Positive bending	1,060 or 1,560	1,350	1,350
Negative bending	1,200 or 1,700	1,350	1,350
Diagonal Tension			
No web reinforcement	108	90	90
With web reinforcement	270	360	360
Bond (Plain Bars)			
Tob bars	85	90	90
Two-way footings	85	108	108
All others	85	135	135
Short Tied Columns (No bending)	850	540	600
Long Tied Columns (No bending)	$p' = p(1.5-0.01\frac{h}{r})$	$p' = p(1.3-0.03\frac{h}{t})$	$p' = p(1.3-0.03\frac{h}{t})$
Reinforcing (Intermediate Grade)			
Tension	17,000	20,000	20,000
Compression	17,000	16,000	16,000

Note: p = allowable load for short column; p' = allowable load for long column; h = unsupported height; $r = \sqrt{I/A}$; and t = least dimension. The maximum extreme fiber stresses given in the Brazilian code can be used when all factors affecting stress are considered.

The effect of these design stresses on the size of columns is demonstrated in Fig. 4. If a short column built of 3,000 psi concrete and axially reinforced with 5 sq in. of steel should be subjected to a load of 200,000 lb, the ACI code would require this column to be 15 in. square, whereas the Brazilian code would require it to be only 12 in. square. Under the compromise stress used in the design of the extension it would be 14 in. square.

The Brazilian code also allows an increase in concrete compressive stress exceeding 40% for members in flexure if the concrete mix is controlled, if it complies with ultimate stress requirement, if all the forces acting on the structure are considered in the design, and if the design materials and workmanship





NOTE:
LOAD OF 200,000 LB
REINFORCEMENT 5 SQ. IN.

FIG. 4.—COMPARISON OF COLUMN SIZES

are adequately inspected. However, under some conditions, the applied forces over the normal dead and live loads may be so insignificant that the use of the increased allowable stress results in extremely slender members, which have insufficient factors of safety by United States standards under constant loading conditions. It is realized that the ACI code working-stresses were established to allow for extensive variance in climatic conditions, such as occur in the United States, and that these conditions will not be so severe in Brazil.

TRANSPORTATION

One of the major problems encountered in the construction of the São Paulo Piratininga plant was transporting heavy and bulky equipment to the site, such as turbine generator parts, boiler drums and transformers. The city of São Paulo, with a population of about 3,000,000, is on the plateau at an elevation of approximately 2,300 ft and a distance of approximately 60 km inland from the coffee port of Santos. It is approximately 500 km from the port of Rio de Janeiro. Its approach from Santos is by three railroad lines and the new Via Anchieta Highway and from Rio de Janeiro by the Central Railroad and a modern highway. The highway bridges on these routes were found to be inadequate for carrying the larger parts by truck trailer. From the nearby port of Santos, the railroad tunnel clearances and load limitations did not permit transportation of all the heaviest and bulkiest parts; therefore, these parts were shipped through the port of Rio de Janeiro.

The stators for units 1 and 2 were the heaviest single pieces to transport, weighing as shipped approximately 260,000 lb each. These were shipped as deck loads to the port of Rio de Janeiro and, as there was no crane available of sufficient capacity to unload them, were skidded off the ship side from the deck loaded position. Here they were loaded on a well car, available in Brazil, prior to shipping. All bridges along the Central Railroad line from Rio de Janeiro to São Paulo were checked and tunnel clearances confirmed by a template run. To obtain the necessary clearances at all points, it was further necessary to chip rock in some tunnels and move tracks and many signal obstructions. After reaching the railroad yards in Sao Paulo, the stators were transferred to a specially prepared trailer on which they were transported to the site. Since this procedure required painstaking study, the transfer was made without incident.

The stators for units 3 and 4, each completely assembled, would have had a shipping weight of approximately 385,000 lb each, which is 125,000 lb more than the weight of the units 1 and 2 stators. This weight exceeds the limits of any railroad or trailer equipment available in Brazil. Although the Central Railroad bridges have been generally strengthened for E-72 loading and could carry this load when placed upon a special railroad car, it would have been necessary to construct a special well car to carry the completely assembled stator. It was established as more economical to ship the shells of the stators without their windings and to install the windings in the field. This reduced the weight of each piece to approximately 166,000 lb and, therefore, it was possible to unload the stators from the ships using the floating crane equipment available in the Rio de Janeiro harbor and then to transport these stators in the same manner as the stators for units 1 and 2.

As a note of interest, these are the first large stators to be stacked and wound at the site using factory procedures and tests. This will probably lead

to more large units being used in remote areas where transportation of heavy pieces is a problem.

Generally, as much of the equipment as possible was transported through the nearby Santos port and by rail or highway up the Serra to the plant site.

A staff of key supervisors in the various crafts was furnished from the United States, but all skilled and unskilled labor was obtained locally. The labor was adequate in amount and eager to learn, but considerable training was necessary to obtain men qualified for pipe welding and other highly specialized phases of a job of this magnitude. Total man-hours for this work is appreciably higher than for a comparable job in the United States. This is partly due to the lack of experience and the lack of plentiful modern construction equipment. However, the labor rates being comparatively low, the over-all costs of the work are comparable with domestic projects.

With the completion of the fourth steam unit at Piratininga and the installation of the additional units at Cubatão, the São Paulo system will have a gross capacity of 1,369,676 kw, of which 410,000 kw will be at Piratininga.

Journal of the
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HYDROELECTRIC POSSIBILITIES AND INFLUENCE OF LOAD GROWTH[a]

By S. P. McCasland,[1] F. ASCE

SYNOPSIS

This paper is devoted to the physical and economic enhancement that is imparted to hydroelectric potentialities by expansion of the power markets to which they are accessible. Particular emphasis is given to the improvement which small undeveloped sites may derive from the rapid increase in demand which is being encountered by many young power systems in foreign countries.

It is widely known that the power potential of many great rivers, such as that of the Congo with 25 million kw, at Leopoldville, cannot be harnessed because of the minimum size of the projects discourages their use their the present. The fact that many potential hydro projects remain too small to be attractive until the accessible market grows to sizeable proportions, on the basis of steam-generated power, is not widely appreciated. In some cases, a site which might have contributed only a nominal, dependable capacity to the load of the early 1920's, may become capable of a substantial contribution to the present-day load. As explained in detail subsequently, the larger load offers a shorter peaking period for minimum-stream flow and, thus, increases potential firm capacity; a bigger load can also absorb more hydro energy at times of high-stream flow. Outstanding enhancement is not, by any means, imparted in all cases of load growth. Occasionally, however, a site which was quite properly rejected long ago, as too small to be of interest, may be found to have attained very substantial capability. However, such increased hydro

Note.—Discussion open until September 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. PO 2, April, 1960.

[a] Presented at the October, 1959 ASCE Convention in Washington, D. C.

[1] Chf. Hydroelectric Engr., Ebasco Internatl. Corp., 100 Church St., New York 7, N. Y.

capability is of little importance to the power consumer, unless it makes possible the production and sale of hydro power at a rate cheaper than some other source, such as a steam boiler.

Stream flow and available head have value only in relation to the power system into which the hydro output can be incorporated, and the demand which it can serve. For any given load, a particular site—with definite head and water supply—is potentially capable of a certain contribution in both capacity and energy. With any other load, however, this potential capability would be different; in other words, capability is a function of load as well as of water and head. If a plant is not constructed with enough generating capacity, the potential capability may not be fully realized; but regardless of the magnitude of hydro generators, no more than the potential capability can be attained for the same combination of load, water, and head. No engineer would design a bridge without knowing the load, and this knowledge is just as essential to a hydro analysis, even though the term "load" means something quite different in the latter case.

In considering prospects for the development of available head and water for a particular load, the engineer is evidently faced with the necessity of comparing hydro cost with that for the cheapest alternative means of power production. In practice, this usually means a comparison of hydro cost with that for steam or diesel-generated power. In the areas under discussion there is usually no opportunity to incorporate auxiliary benefits, such as flood control, and the hydro power must stand on its own merits. As inferred previously, there is usually no reason to undertake a hydro development if it does not have a cost advantage. A hydro project ordinarily entails a heavy initial outlay for civil engineering works because the dam, tunnels, etc., must be built to ultimate size at the outset, even though the installation of some of the generating units may be postponed. Also a transmission line must be built from the hydro site to the load center. Although the incremental cost of adding future units may be small, and the project's final average cost per kilowatt may be correspondingly low, interest on the early investment, during the years before the complete installation is needed, may be a critical factor. For example, it might be decided that X River could be developed to meet load growth from 1965 to 1980, by the progressive installation of three 10,000 kw units in 1965, 1970, and 1975. It might be estimated that the final investment in the ultimate project would be $6,000,000, which might seem to be quite a bargain, being only $200 per kw. But it might cost $5,000,000 to get the first unit on the line, even though the other two would come along for $500,000 apiece when they would be needed. Thus, the project would be saddled with interest on some part of $5,000,000 for 15 yr before the investment would be fully productive. With money at 10% or even 12%, which is not unusual for foreign installations, interest could be quite a factor in actual cost. Thermal additions, on the other hand, have a more uniform cost per kilowatt, and consequently entail a more gradual rate of investment with a lesser penalty in interest charges. Particularly for small power supplies, the initial hydro expenses may be prohibitive and for many young systems, thermal generation has been clearly the best course from the standpoint of both the utility and its customers. With expansion of the system by such thermal additions, however, a small and formerly unattractive hydro site may have become sufficiently enhanced to be as competitive as the next step in extension of power production facilities. For example, suppose the development of X River had been postponed for 30 yr. The capability of the site might be increased to 50,000 kw under a load sufficiently large to call

for the full 50,000 kw in a single step at the outset. Obviously the hydro plant would be more attractive under the latter conditions.

There are factors outside the fields of economics and engineering which influence the development of power systems, and these are sometimes accentuated in foreign countries. An example of the peculiar situations which can develop may be of interest. In some parts of the world, the family is still about the largest unit of private business because social custom does not afford a ready opportunity for the formation of local corporations or even partnerships on a large scale. In addition, there are factors which frequently discourage the entrance of interests from outside the country. In these circumstances, one prominent middle eastern city of about 1,000,000 people came to be served, in the mid 1950's, by more than twenty-five independent power systems, each covering a segment of the city. There had been no entity with enough resources to cope with the load as a whole, and most of the power systems simple represented the limit of family finances. In most instances power was scarce and had to be rationed, even though it was customary for the rate per kilowatt-hour to ascend with greater consumption, instead of descending in the usual manner. There, and in some other parts of the world, power has been regarded very largely as a means of providing light. A sizeable block of customers on such systems still may consume only about 10 kwh a month. Plants are apt to be shut down during part or all of the daylight hours, and rates up to $0.20 per kwh, or even more, are not unknown.

While demand is still subject to these "growing pains" in some parts of the world, the situation has changed rapidly since World War II. In most foreign areas where service has been good and rates reasonable, the post-war increase in demand has been phenomenal. There has been an increased interest in appliances such as stoves, irons, and air conditioners, and there has also been a pronounced acceleration in the trend of industrialization. Industrialization provides factory employment, servants become harder to get, and appliances become more popular. In some areas of the Americas, compound annual-load growth of 12% or even 14% has occurred. At such rates, even a small market can become a big one in an astonishingly short time, and sometimes formerly discarded hydro sites can attain importance, as a result.

In considering the power which can be derived from a hydro site, it is all-important to differentiate sharply between potential capacity and potential energy; that is, between kilowatts and kilowatt hours. With the vast amount of interconnection, diversity, and reserve that exists in the United States, it is often the custom to wholesale power on the basis of energy alone, and there is sometimes a tendency to lose sight of capacity value. But the utility that distributes and retails the power can never be indifferent to this vital factor, and the wholesale price always reflects it. Particularly in a small isolated system overseas, where a single hydro plant can be a substantial component, its contribution to capacity is a matter of physical necessity. If the hydro plant does not measure up to expectations on some dry day, a part of the load has to be rejected with all the undesirable consequences of injured public relations, criticism in the press, and perhaps even real damage to some customers. A shortage of energy, on the other hand, simply means that some steam plant has to burn more fuel for a time, with a strictly temporary debit on the books. Further comment as to energy production is offered subsequently, but first consideration will be given to capacity, which is by far the most important component of demand in the areas under discussion. The capacity which any particular hydro site can contribute to a given load with reliability is known as

"firm" capacity. It is governed by the water supply that will be available in the greatest drought reasonably expected, that is, the driest day of the driest year, or if there is storage, the extent of the driest period likely to occur. An estimate of firm capacity thus calls for a determination of minimum-water supply, and this is one of the most troublesome problems in foreign hydro work. Although there are outstanding exceptions, satisfactory records of stream flow are not usually available. In some cases, existing records even have been thrown away by newly appointed officials who did not appreciate their value. When a private or public entity becomes especially interested in a particular site, it is, of course, usual to gage the water. But such keen interest is apt to develop only when the site comes under active consideration, and not decades earlier. The idea seems to prevail, even among engineers in other fields, that hydro engineers have some sort of crystal ball for determination of water supply. In the United States there might be a 50-yr record, and the sponsor of a project would have, as a calculated risk, at least an even chance that no worse conditions than those recorded would occur during the economic life of the plant. But if there had to be 50 yr of records, there would be very few hydro plants in the areas under discussion. One has to use whatever data he can get. Sometimes the projects are under-built because of a too-conservative estimate.

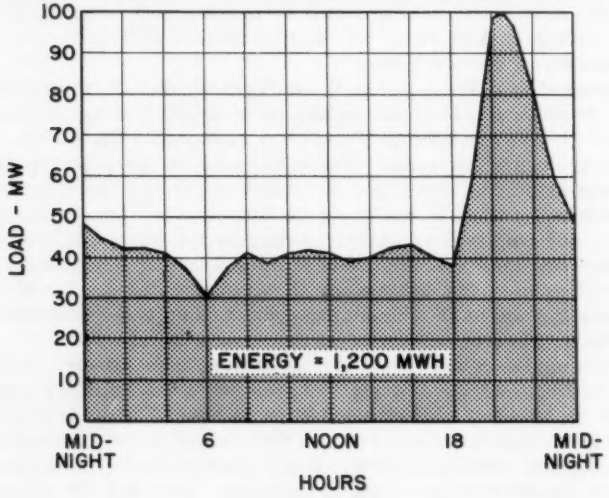

FIG. 1.—TYPICAL DAILY LOAD CURVE, 50% LOAD FACTOR.

Conversely, they may be over-built. Sometimes the operators can fall back on reserves in time of drought. And sometimes there is a power shortage. After the installation of a hydro plant, however, further load growth is often met by thermal additions; this usually improves the hydro capability and the significance of water shortage may diminish considerably. An unexpectedly critical drought may not occur for many years after the project goes into service, and during that time the hydro capability may be strengthened enough to contribute its full installed capacity, even with a lesser minimum water supply than anticipated.

The basis for improvement in potential-power capability is illustrated graphically by Figs. 1 through 5. Fig. 1 shows a typical daily-load curve for a small system. The ordinate of the curve represents the demand on capacity,

and the abscissa, time; the area obviously is energy. The relation of energy to capacity is expressed by the "load factor," which may be defined as the ratio of average load to maximum load; this factor is considerably lower in young systems than in the United States. In the case illustrated, the load factor is about 50%. At 6 A.M. demand is at the minimum for the day, 30 mw; this is called base load. At about 8 P.M. the peak demand of 100 mw occurs.

For illustration, suppose the load is met by four identical 25 mw steam generators as in Fig. 2. On the day illustrated, the four generators would have to produce a total of 1,200 mwh of energy. One, labelled "D", would operate continuously on the base of the load and would have to generate half this amount, namely 600 mwh. However, generator "A," operating on the peak, would be in service for only about 3 hr and it would be operating at full capacity for only a small part of that time. It would be called on to produce only 48 mwh. Although this generator constitutes 25% of the required capacity, it must produce only 4.0% of the required daily energy, and its boiler will consume less than 10% of the fuel needed in the case of generator "D." Now let us move 5 yr into the future, and assume that the load of each hour has doubled, with two 50 mw units

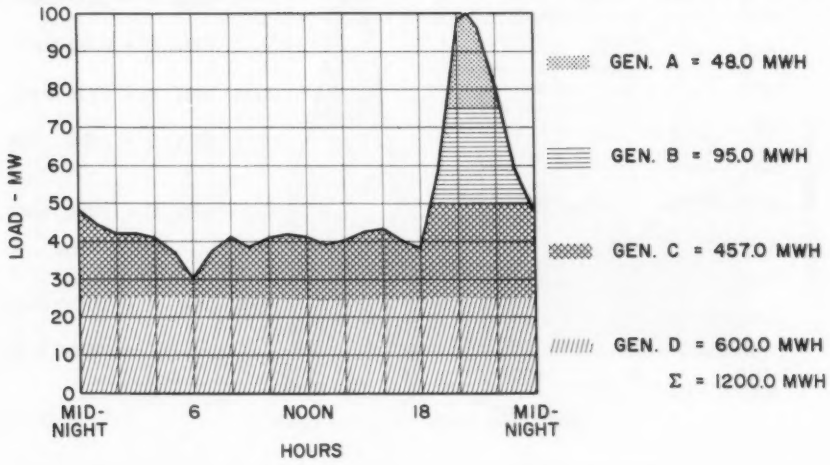

FIG. 2.—ALLOCATION OF PLANTS, 100 MW DAILY LOAD, 50% LOAD FACTOR.

added to the system. The increased load is shown by Fig. 3. Generator "A" now is only 12 $\frac{1}{2}$% of the total capacity and it need produce only 32 mwh. This is two-thirds as much energy as the same generator was required to produce 5 yr before, and only 1.3% of the daily energy in the larger load. This relation between percentage of capacity and corresponding energy, expressed by what may be called a "peak percentage" curve, is shown by Fig. 4. The scales are expressed in percentage and the curve applies to the loads of both Figs. 2 and 3; that is, to any magnitude of load having the same hourly variation. For a load which increases from year to year, a fixed number of kilowatts on the peak is obviously a smaller and smaller percentage of total operating capacity. The essence of the matter, however, is that this diminishing percentage of capacity is called on to produce a disproportionately smaller percentage of daily energy. In other words, the required percentage of energy production decreases at a

much faster rate than the percentage of capacity. Note that 25% of the peak capacity produces 4% of the daily energy; 12 $\frac{1}{2}$%, 1.3% of the energy; and 5% only 0.3% of the energy.

Suppose Generator "A" had been a hydro unit. With the head available, the water supply on the minimum day represents a definite amount of daily energy. The stream flow can be accumulated in a small pond during off-peak hours and the total daily water supply can be released through the plant at will during a short peaking period. Generator "A" would have to produce 48 mwh in order to meet 25 mw of the 100 mw load as shown by Fig. 2. However, with the load increased to 200 mw as shown by Fig. 3, Generator "A" would be only 12 $\frac{1}{2}$% of the total capacity and it would be required to turn out only 32 mwh to meet 25 mw of the increased load; 32 mwh obviously can be produced with only two-thirds of the water required for the production of 48 mwh. Thus, doubling of

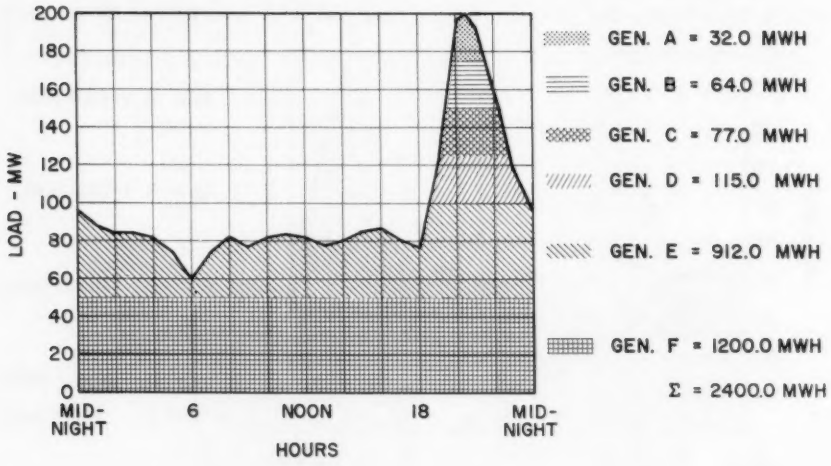

FIG. 3.—ALLOCATION OF PLANTS, 200 MW DAILY LOAD, 50% LOAD FACTOR.

the system load makes it possible for the same plant to maintain its full capacity with two-thirds of the water required in the case of the smaller load.

It can also be shown that the same site with an undiminished water supply—that is, one capable of producing 48 mwh in a day—is potentially capable of a greater contribution than 25 mw to the larger load. As shown by Fig. 4, 48 mwh is 4.0% of the daily energy in the 100 mw load, and, obviously, it is half that percentage, or 2.0% of the energy in the 200 mw load. From Fig. 4, it can be seen that the latter energy percentage will sustain 17.0% of the required peak capacity, or 17.0% of 200 mw. Thus, doubling the load increases the potential, dependable hydro output from 25 mw to 34 mw with the same water supply, and the same site.

Fig. 5 illustrates an extreme case of how hydro capacity can increase with the same site and water supply. This hypothetical site offers 100 m of head and a water supply averaging 1.0 cms, or about 21 mwh on the minimum day.

At the top of Fig. 5 there is a curve which illustrates the utilization of this energy at 60% load factor for an isolated load, where the one hydro plant would be the only source of power. Under such conditions the plant would have to be in operation for 24 hr a day and the maximum demand that the hydro plant could meet would be only 1.4 mw. It might not be worth while to build the hydro plant for such a small output. However, if steam additions were made to support load growth to the extent shown by the lower curve, the situation would be different. Here the hatched area "II" at the base of the curve represents the thermal energy produced by 40 mw of thermal capacity in extreme drought for a load of 50 mw having the same characteristics as above. With this thermal support, the hydro plant would be in operation for only about 3 hr, and the same minimum day hydro energy—21 mwh—could meet 10 mw of the load as shown by the

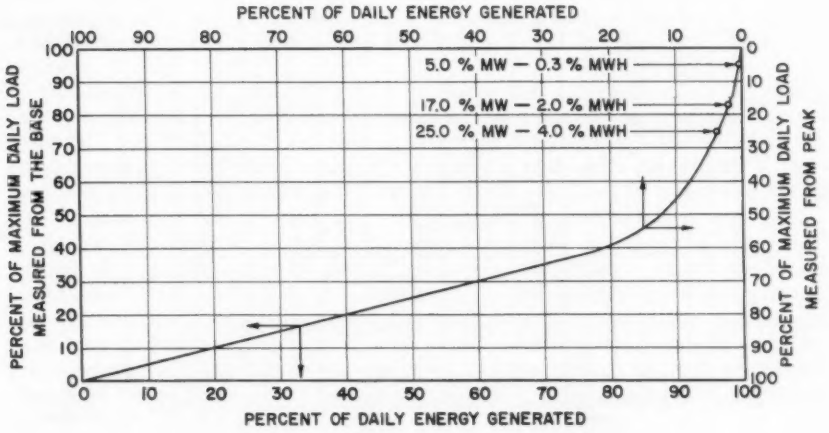
PERCENT OF DAILY ENERGY GENERATED
100 90 80 70 60 50 40 30 20 10 0
5.0 % MW — 0.3 % MWH
17.0 % MW — 2.0 % MWH
25.0 % MW — 4.0 % MWH
PERCENT OF MAXIMUM DAILY LOAD MEASURED FROM THE BASE
PERCENT OF MAXIMUM DAILY LOAD MEASURED FROM PEAK
0 10 20 30 40 50 60 70 80 90 100
PERCENT OF DAILY ENERGY GENERATED

FIG. 4.—PEAK PERCENTAGE CURVE, 50% LOAD FACTOR.

hatched area "I" on the peak of the curve. In other words, this latter area has exactly the same size as that under the upper curve, but due to the shorter operating time with the larger load, the attainable hydro capacity is seven times the hydro capacity attainable in the situation shown by the upper curve. The hydro development might very well be justified on the basis of this larger output.

Improvement in hydro capability, as described above, is more pronounced with low-load factors and sharp peaks. It sometimes happens that load factors increase as the magnitude of the peak load increases, and also the peak may become more blunt. These conditions can diminish, and in extreme cases even entirely erase the gain in hydro capability. In most cases, however, the shape of the load curve tends to change slowly in relation to load growth. This is particularly true in young systems.

Up to this point, comment has related to the contribution of the hydro plant to system load-carrying capacity under the worst water conditions. In years

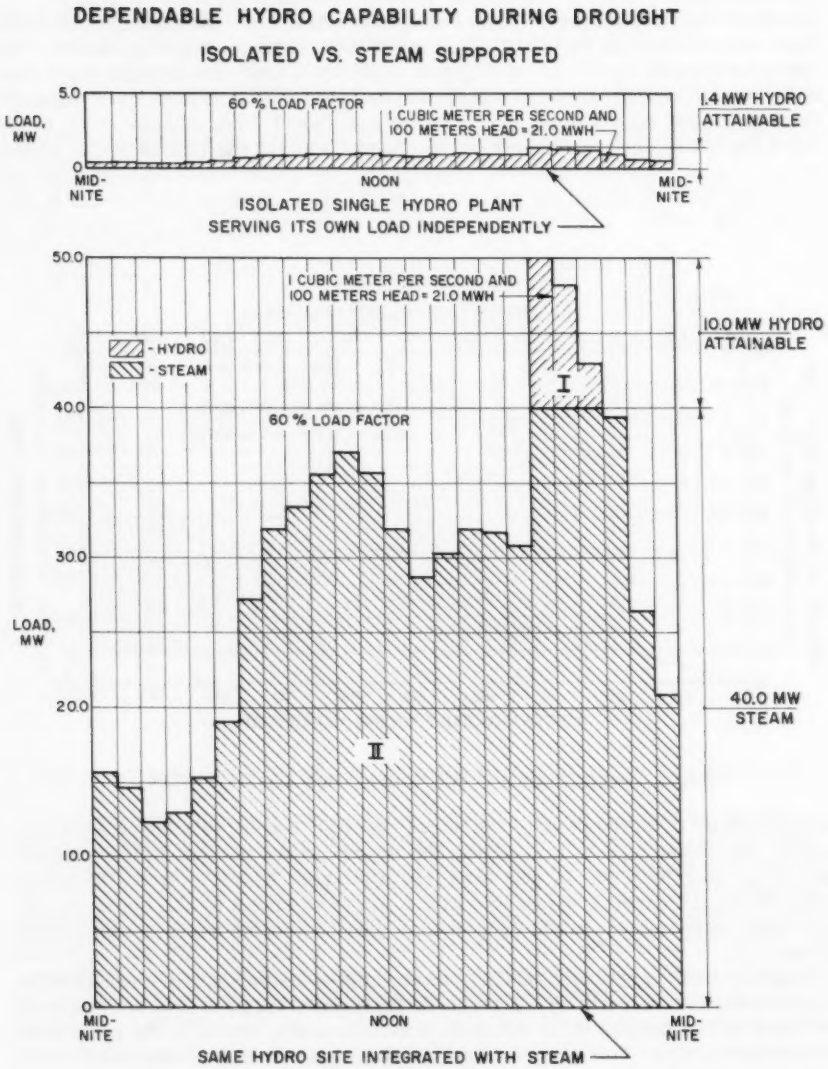

FIG. 5.—DEPENDABLE HYDRO CAPABILITY DURING DROUGHT. ISOLATED VS. STEAM-SUPPORTED. SAME HEAD-SAME WATER. DAILY LOAD CURVES AT 60% LOAD FACTOR.

of critical drought, a hydro plant may produce very little energy and in a combined hydro-thermal system most of the energy component of demand probably will be produced with fuel. However, years of great drought are rare. In wet years a hydro plant may operate continuously in the position shown for unit "D" in Fig. 2, and produce half of the system energy. At such times thermal units are switched to the peak, where they contribute their full capacity with little fuel, just as hydro units on peak require little water. In normal years, hydro might occupy some intermediate position between the base and the peak as, for example, that for unit "B" or "C" in Fig. 2. Over the life of the hydro project, the quantity of hydro energy which can be produced is a function of the average water supply, and not the minimum which governs capacity.

A hydro plant's energy falls short of full use of the average water supply under two conditions. The first of these conditions exists when the uncontrollable stream flow exceeds the maximum volume that can be passed through the turbines, and spills past the plant intake. To convert such spill to useable energy, additional plant capacity may be installed. However, this additional capacity is inoperable in dry years and hence cannot help to meet the load. In some cases, such excess capacity can produce enough energy, on the average, to justify the cost of its installation only by the fuel saving at the thermal plants. Also load growth and thermal additions may ultimately "firm up" the excess capacity.

The second occurrence of waste is in water that can produce hydro energy only during hours when there is no demand. For example, suppose both generators "C" and "D", Fig. 2, are hydro units operating during a season of an uncontrollable stream flow that is sufficient to run both units continuously. During the hours from midnight to 6:30 P.M., there would not be a demand for all the output of unit "C", consequently there would be some waste of water.

As the load increases, potential energy production is improved by the possibility of reducing waste in both of the above categories. First, the larger load makes practicable a bigger installation, consequently there are, potentially at least, fewer occasions when the turbines will reject water. Second, the base load increases along with the peak load, and this eases the off-peak restriction on the marketability of energy. Notice that for the 200 mw load of Fig. 3, 50 mw of hydro could be run continuously, while this was impossible in the 100 mw load of Fig. 2.

CONCLUSIONS

Hydro power must be competitive in cost with power from other sources, to be attractive to the consumer. The value of a hydro site depends just as much on the system into which its output can be incorporated and the demand it can serve, as it does on water supply and fall. A measure of the project's worth is, thus, the steam or diesel expense which can be avoided by its construction. The firm hydro capacity which can contribute with reliability to meeting the load makes it unnecessary to build equivalent thermal capacity, and the marketable hydro energy which can be produced renders unnecessary the fuel cost and other operating expenses incidental to producing the same amount of energy by steam. If the present worth of estimated annual charges on the hydro plant—that is, the capitalized value of interest, amortization, taxes, insurance, and operation—is less than the present worth of similar charges and fuel expenses corresponding to the thermal power avoided, the hydro is the best buy. And even if the hydro is uneconomical today, it may become attractive at some future time.

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ROCKFILL DAMS: BROWNLEE SLOPING CORE DAM[a]

Discussion by Vilgot Lanner
Closure by Torald Mundal

VILGOT LANNER.[1]—Mr. Mundal's paper gives a comprehensive and valuable engineering picture of the 400 ft high Brownlee sloping core rockfill dam. This paper was of special interest to the writer since his organization is presently engaged on the Trängslet Dam of similar type, height and yardage. Brownlee and Trängslet Dams are all of sloping core rockfill type, of approximately 400 ft height and each containing approximately 10,000,000 cu yd of material.

At the turn of the year 1959-60 the Trängslet dam had reached 75% of the total height; and storage was 250 ft. The entire project will be completed at the end of 1960. It is the intention of the writer's organization to make a detailed presentation of the engineering and early performance of the dam in 1961. Hitherto, only very brief international mention of the Trängslet Project has been made, viz. in the Engineering News-Record, Sept. 11, 1958, pages 60-66, (special issue on the occasion of the Sixth International Congress on large dams—ICOLD—in New York). However, the dam was demonstrated to the participants in the 26th Executive Meeting (ICOLD) on June 11, 1959. The entire project is planned, designed and constructed by the private company Stora Kopparbergs Bergslags AB., Falun, Sweden, under the management of Vilgot Lanner with Thorild Persson as chief designer, and Albert Andersson as chief resident engineer. During the visit to Trängslet printed information on the project was handed out to the participants. The writer presents herewith information concerning the dam in order to put technical data on these high rockfill dams together in the technical literature as a part of this very worthwhile Symposium. Rather than compare features of the dams, the writer will present data on Trängslet Dam and leave comparisons up to the reader.

GENERAL

The local conditions are such that the dam at Trangslet could be constructed either as a concrete gravity dam, or as an earth or rock fill-type dam. The fill-type of dam was decided on, in view of greater permanence. Since material was readily available near the site, a rockfill dam with impervious core was found to be technically and economically motivated. An inclined impervious core was given preference to a central core for the following reasons:

1. With an inclined impervious core it is possible to carry out the supporting rockfilling in advance, and this involves that initial settlements can develop

[a] June, 1958, by Torald Mundal.

[1] Vice Pres. and Mgr., Power Dept., Stora Kopparbergs Bergslags Aktiebolag, Falun, Sweden.

without the impervious core having reached a point where it would be injuriously affected by such settlements.

2. Construction of a dam with a sloping core provides more regular employment for labour and machines than a dam with a central core in a climate where there is a limited season during which core material may be placed.

3. With an inclined core, the downstream rockfill may be sluiced with water without risk of damage to the filters.

At Trangslet the rock consists of syenite and porphyry with isolated sections of diabase. The surface layer of foundation rock had been affected by atmospheric decomposition, and extensive clearing operations were necessary, especially in the area covered by the impervious core.

Conditions at the site are such that the cost per cubic yard for rockfill and moraine will be approximately the same. This, together with the fact that the shear strength of the core material is high, has contributed to the decision that the impervious core should be relatively thick. A general view of the dam is shown in Fig. L1 and a cross section in Fig. L2.

In order to allow for future settlements, the impervious core is horizontally bow-formed with a maximum chord of 50 ft, and will, when completed, rise above the crest at the center of the dam by 5 ft over the horizontal plane.

Other principal data:

Height	120 m. (400 ft)	
Crest length	850 " (2800 ft)	
Width of base	550 " (1800 ft)	
Crest width	10 " (33 ft)	
Incline of upstream slope	1:2.5	
Incline of downstream slope	1:1.25	
Volumes		
Rockfill, downstream	(ungraded rock)	3.25 mill. cu m (4.13 mill. cu yd)
Coarse filter, downstream	(moraine stone)	0.50 mill. cu m (0.64 mill. cu yd)
Medium filter, downstream	(graded crushed rock)	0.25 mill. cu m (0.32 mill. cu yd)
Fine filter, downstream	(graded gravel)	0.25 mill. cu m (0.32 mill. cu yd)
Impervious core	(graded moraine)	1.50 mill. cu m (1.91 mill. cu yd)
Filter, upstream	(ungraded gravel)	0.25 mill. cu m (0.32 mill. cu yd)
Supporting rockfill, upstream	(ungraded rock)	1.20 mill. cu m (1.52 mill. cu yd)
Total		7.2 mill. cu m (9.2 mill. cu yd)

FIG. L1.—GENERAL VIEW - CONSTRUCTION OF TRANGSLET DAM

Construction.—Work on the base rock.—All loose and decomposed rock must be removed from the area where the impervious core and the filters will contact the base rock. The surface of the rock is then cleared and concrete is added in order to obtain a continuous contact surface. To prevent ground water damaging the impervious core at the contact surface, the ground rock is sprayed with concrete, thus sealing all cracks.

Below the impervious core and in a position indicated in Fig. L2 a grout curtain has been made in the rock. The depth of these cement injections is determined by drilling tests and pressure measurements in drilled holes at various depths. Generally, injections have been made to a depth of about 65 ft (20 m).

Some 65 ft (20 m) downstream from the injection curtain and at a depth of about 65 ft (20 m) an inspection gallery runs under the entire core, acting as a drain for the rock under the core. In the future, any necessary complementary injections under the dam can be carried out from this tunnel. The inspection gallery is drained into the discharge tunnel.

Fill Materials.—The rockfill consists chiefly of porphyry from the quarry upstream from the dam. Rock from the various underground operations—alternatively porphyry and diabase—is also put in the dam. In general, tunnel rock proved less suitable as rockfill than the other types of rock, by reason of the relatively high proportion of small-sized material.

Rock size:	<500	<200	<100	50 mm
Quarry rock	70	40	22	10 %
Tunnel rock	99	82	55	23 %

For this reason tunnel rock is normally used in the filters after crushing and grading.

The filter between the impervious core and the rockfill comprises three layers, viz. coarse, medium, and fine. The rock in the coarse filter consists

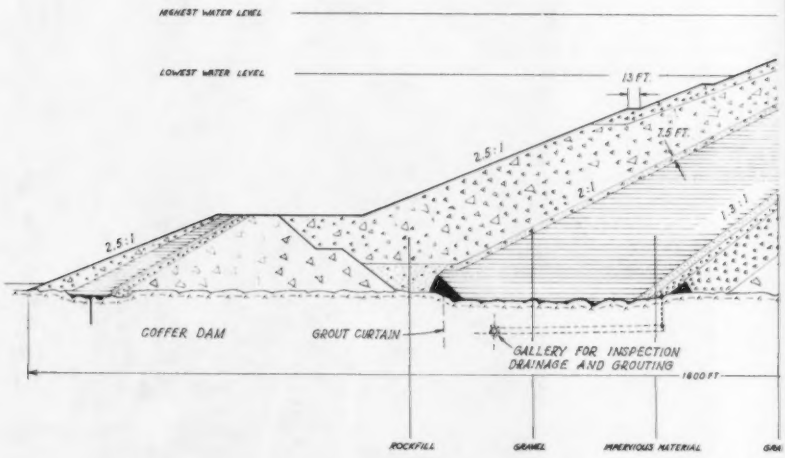
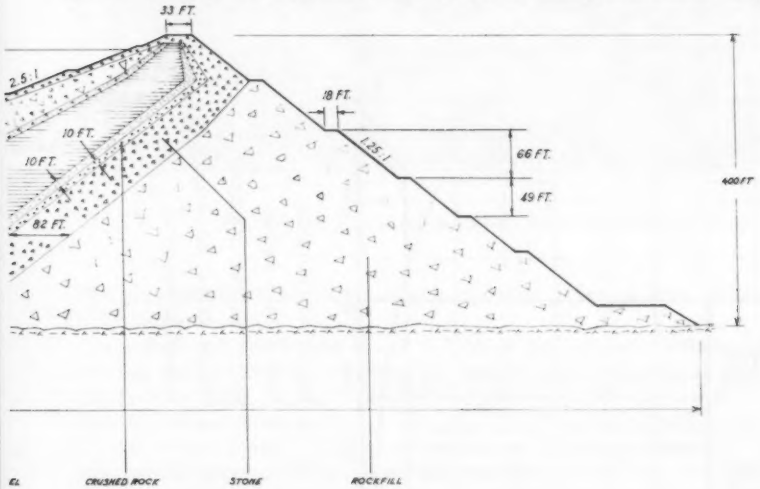


FIG. L2.—CROSS SECTION

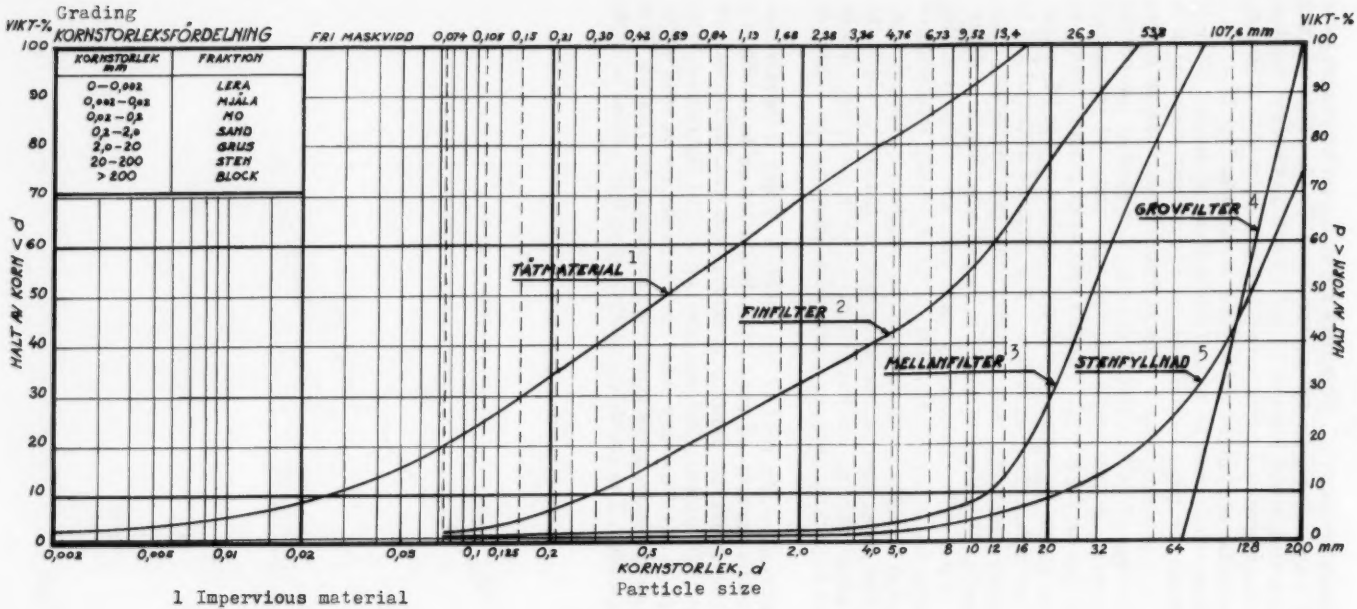
of stones out of the moraine material. This rock normally varies in size from 100 to 200 mm and is very suitable for the coarse filter. Graded crushed rock from underground operation, sized between 15 and 80 mm, is used for medium filter. The fine filter is composed of river gravel to a maximum size of 50 mm. The grading of the material used in the filters is described in Fig. L3.

Material for the impervious core is taken from a moraine deposit close to the dam site. To render the moraine suitable for use as core material it is necessary to reduce the content of stone. This is done by rejecting all pieces over 100 mm. An average of 30% is sorted out and then used in the coarse filter, as described above. In this way a suitable core material is obtained with a clay content of approx. 3% ($< 2\mu$). Further details of grading appear in Fig. 3. The core material has its optimum moisture content at 7 to 8%, calculated on the basis of material less than 16 mm. The moraine material has in general a permeability considerably lower than the value established as an upper limit, i.e. 0.10 cm/h. The shear strength of the material has been determined and the value of ϕ is 45°. Laboratory tests have also shown that this value will increase when the reservoir is filled. This is equally true whether the material is packed at Proctor optimum or by the wetpacking method. Laboratory tests have also been carried out to determine the effect which the variation of storage would have on the stability of the core material. These tests disclose that the variations have no appreciable effect on the degree of settlement.

Placing of Fill Materials.—The part of the rockfill, which transmits the load from the impervious core to the base rock, is sluiced so as to obtain a body of sound supporting material. Sluicing is carried out with high-pressure nozzles at a pressure of 20 atmos. (300 psi), the capacity of the pumps being 6000 l/min (1320 gal per min). To every cubic yard of rockfill 4 cu yd of water are used. Rockfill downstream from the supporting section is not sluiced. This part is built up during winter. The rockfill is dumped from fairly great heights,



TRANGSLET DAM



1 Impervious material

2 Fine filter material

3 Medium filter material

4 Coarse filter material

5 Rock-fill material

FIG. L3.—GRADING CURVES - TRANGSLET DAM

up to 20 m (65 ft) as experience indicates that settlement will be less when such heights are used. This applies particularly to rockfill that is dumped in winter.

The coarse filter is sluiced and then packed by tractors in a layer of 0.5 to 1 m thickness. The medium and fine filters are sluiced and packed in layers of 0.5 m thickness. The vibrator used in compaction is shown in Fig. L4.

The moraine material is compacted in 0.3 m loose layers (11.8 in.) by 40-ton rubber-tired rollers. Six rollings give over 95% effective compaction which is the minimum aimed at. During rolling, the moisture content is kept, if possible, around 7.5 to 8%, or somewhat over the Proctor optimum. The material is dampened when necessary. Experience shows that it is difficult to drive trucks and rubber-tired rollers over the material if the moisture content goes up to 10%. At a moisture content of 10.5% such operations are not practicable.

FIG. L4.—VIBRATING THE FILTERS TRANGSLET DAM

The photograph of Fig. L5 shows a summer stage of core, filter and rockfill construction.

Control.—A continuous check is kept on selection and treatment of material. Special care is exercised over the quality and treatment of material for the impervious core. In the course of compaction, tests are made of the comparative degrees of compaction. A sample of such a test report is shown in Fig. 6. A modern aid to the continuous and rapid checking of packing is the isotope test, which gives quick and reliable results.

Observations.—The dam, during construction as well as for a long period after completion, will be the subject of various observations, including measurements of settlement, leakage and pore pressure.

Settlement.—As the successive stages of the dam are completed, settlement will be observed from a number of points at different levels. Measurements carried out hitherto have been mainly concerned with material placed in winter and thus not sluiced. There is evidence that settlement decreases rapidly as time goes on.

Leakage.—Leakage along the entire dam is expected to amount to 8 l per sec (1.8 gal per sec.) with a seepage rate of 0.10 cm per hr (0.04 in. per hr). Permeability has been found to be better than anticipated and the actual leakage

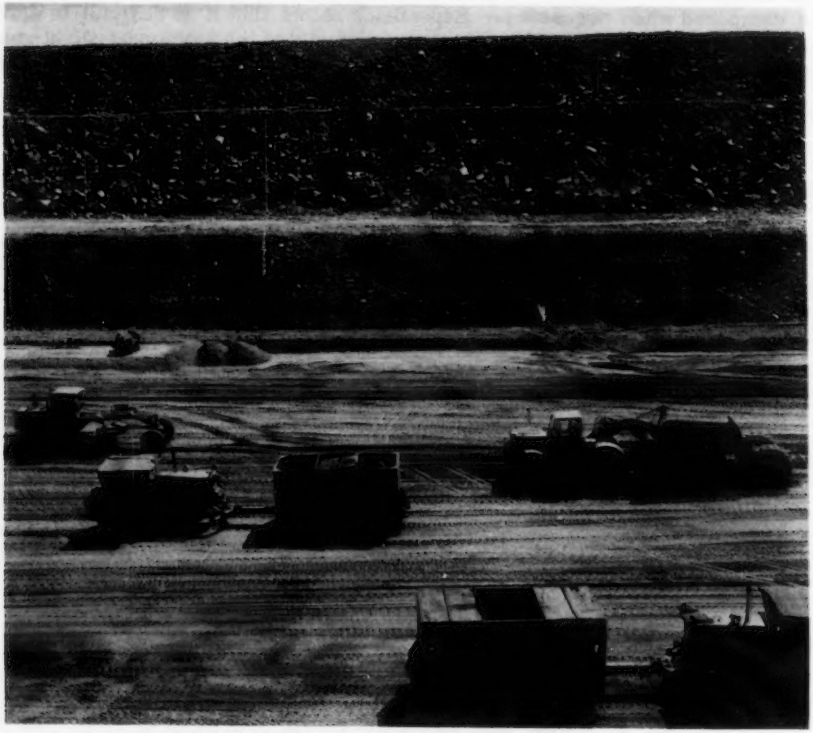


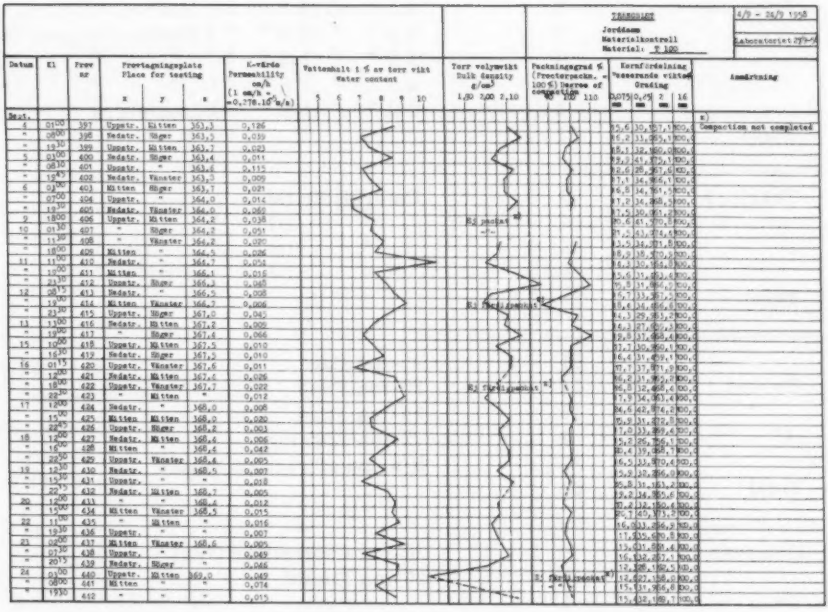
FIG. L5.—WORK ON THE CORE IN EARLY SUMMER 1959 - TRANGSLET DAM

might therefore be less than the value given. Leakage is measured at three different points:

1. A spillway has been arranged at the foot of the dam, in which all water is collected and measured. Correction for rainwater snow will be necessary.
2. Under the lowest section of the dam there is a gallery in which leakage water from the contact zone between the impervious core and the ground rock is collected and measured. This arrangement is already in use. At the present storage level upstream, which is 35 m (115 ft.), the leakage does not exceed 0.2 lit./sec. (0.044 gal.).

3. The rock under the core drains into an inspection gallery running under the entire length of the core. Measurements of leakage water are taken in vertical drainage wells located every 5 to 10 m (16.4 - 32.8 ft.) from each other.

Pore Pressure Measurements.—Instruments are being introduced into the interior of the core, to enable pore pressures to be measured at different points in the body of the dam. Similarly, pressure measurements are made in fissure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Grading
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Remark
Compaction not completed
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Mitten
Nedstr.
Höger
Vänster
Mitten
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		Winter (6 months)	Summer (6 months)
Rockfill	(daily)	6.000 cu m (7.600 cu yd)	3.000 cu m (3.800 cu yd)
Filters (all)	"	-	3.000 cu m (3.800 cu yd)
Core material	"	-	5.000 cu m (6.400 cu yd)

CONSTRUCTION EQUIPMENT

The most important equipment used is:

Excavators	3	4.5 cu m (6.0 cu yd) Marion
	2	1.5 " " (2.0 " ") Ruston-Bucyrus 38
	1	1.0 " " (1.3 " ") " " 30
	1	0.5 " " (0.6 " ") " " 19
	4	0.9 " " (1.2 " ") Landsverk
Total	11	
Tractors	6	Caterpillar D8
	4	" D7
	2	" D6
	1	" D4
Total	13	
	11	25-ton Dart
Trucks	14	20-ton Faun
	13	15-ton Faun
	4	7-ton Koering
	20	5/6-ton of various makes
Total	62	
Dumpers	3	20-ton Caterpillar DV21
	1	20-ton Euclid
Total	4	
Rubber-tired rollers	3	40-ton
Vibration machines	5	Bohn und Kähler

Labor.—During the summer half-year, when work is carried on in three shifts, 3 x 110 = 330 men work on the dam. More than 30 cu m per man and day is then brought to the dam, that is, about 4 cu m per man per hr.

Costs.—On the basis of performance hitherto, costs for the various materials are expected to amount to the values shown in the table below.

Material	Length of transport miles	Volume mill cu yd	Capacity cu yd per hr	Unit price exclus. costs for administration £ / cu yd	$ / cu yd	Sw kr / m^3
Rockfill from quarry (blasting loading transport sluicing)	1	6.0	400	6 sh	0.85	6
Coarse filter moraine stone (transport sluicing compaction)	1	0.7	130	5 sh	0.70	5
Medium filter crushed rock from the tunnels (transport crushing screening sluicing compaction)	4	0.35	70	12 sh	1.70	12
Fine filter graded gravel (transport screening sluicing compaction	7	0.35	70	11 sh	1.60	11
Impervious material graded moraine (transport screening compaction)	2	2.0	330	8 sh	1.15	8
Filter	7	0.35	70	8 sh	1.15	8

Mean-price without costs for administration 7 sh/cu yd = 0.95 $ / cu yd = 6.80 kr/m

No foundation work is included in these costs, nor do the calculations include costs for central administration.

TORALD MUNDAL,[2] F. ASCE.—The writer gratefully acknowledges the contributions of Messrs. Leonard, Lawton, Hsu, Ito, Condit and Lanner.

As indicated by Mr. Leonard, the combined cost of excavation and rock in place was unusually low, principally due to the fact that nearly all rock fill

[2] Vice Pres. and Chf. Engr., Internatl. Engrg. Co., Inc., San Francisco, Calif.

was furnished from required excavation. Related factors contributing to the economy of the operation were:

1. Little or no overburden had to be stripped.
2. Blasting was skillfully performed.
3. Excavated rock material required no processing, and there was no waste.
4. There were no long hauls.
5. Because operations were well scheduled, no stockpiling or rehandling of rock was necessary.

Mr. Lawton's question concerning ". . . specially drilled and grouted holes near the left abutment," was well taken. As described in the paper, foundation treatment included both area and curtain grouting in the core trench area. Area grouting was accomplished by means of shallow holes on ten-foot centers each way. Curtain grouting utilized a line of deeper holes. As drilling and grouting operations progressed, it became apparent that the treatment had to be supplemented in the left abutment area. Consequently, a number of additional holes, as much as 300 ft deep, were diamond drilled and grouted to stem the artesian flow. Thus, these holes were "special" in the sense that they were not originally planned; they were substantially deeper than the grout curtain holes; and they had no specific pattern or depth, inasmuch as requirements were dictated entirely by field conditions. Locations and depths were determined by experienced field engineers.

Mr. Lawton suggests additional discussion concerning rockfill sizes and the effects of diversion on the fine and coarse filter zones. With respect to sizes of rockfill, the exposed material at the time of diversion was within the range of 6 to 36 in. As was the case with the impervious core, both the fine and coarse filter zones were at an elevation about 10 ft lower than the downstream rockfill. Since the channel through the rockfill was restricted, the water ponded over the core and filter area, and due to relatively low velocities, silt and cofferdam materials were deposited. After clean-up, it was found that effects of diversion flows were negligible, in that the filter and core zones had not been measurably disturbed.

Mr. Lawton requests justification for the statement, "unit costs for rock placed in the dam is estimated to be about half the cost for similar construction 10 to 15 years ago." An examination of contract unit prices for other rockfill projects constructed since 1945, will reveal that combined costs ranged from $3 to $5. Unit costs for excavation and embankment have, in effect, decreased in recent years because of greatly increased equipment capacities and efficiencies. For Brownlee Project, the factors enumerated at the beginning of this discussion effected economies sufficient to lower combined excavation and embankment operations costs to the extent indicated.

Mr. Condit's discussion of dam movement after completion is of great interest, in that the observations indicate that movement has been somewhat less than what might normally be expected. For example, it is generally felt that settlement of a rockfill embankment can, as a reasonable limit, approach one per cent of the height of the fill. Maximum foundation and embankment settlement at Brownlee was 0.546 ft at Monument F-4 for the recorded period; less than 0.2%. Settlement will undoubtedly continue for a number of years. However, inasmuch as the reservoir has been completely filled since June, 1959, most of the settlement has already occurred, and any future settlement will be at a much lower rate. It would appear, then, that (1) results of foundation in-

vestigations were correctly interpreted, and that foundation preparation and treatment were properly designed and constructed; and (2) the various embankment zones—core, filters, and rockfill—were well selected and well constructed.

Mr. Hsu's presentation of stability analysis details is a welcome addition to the discussion. This information, which was only briefly covered in the original paper, should be of great interest to designers of rockfill dams.

Descriptions of Miboro and Trangslet dams by Messrs. Ito and Lanner provide interesting technical data for inclusion with the discussion. Table 1 presents readily available comparative data for Brownlee, Miboro, and Trangslet. Note that the Brownlee impervious core is substantially thinner than those at Miboro or Trangslet. This may be attributed to the fact that physical characteristics and availability of Brownlee core material were such that design of a relatively thin core could be accomplished.

TABLE M1

	Unit	BROWNLEE	MIBORO	TRANGSLET
Height	ft	400	426	400
Crest Length	ft	1400	1400	2800
Core Thickness (average)	ft	30	93	85
Slopes				
Rockfill, U/S Face	--	1:3.0 (avg)	1:2.5	1:2.5
Rockfill, D/S Face	--	1:1.4 (avg)	1:1.75	1:1.5 (avg)
Impervious Core, U/S Face	--	1:1.5 and 1:1.8	1:1.5	1:2.0
Impervious Core, D/S Face	--	1:1.37	1:0.85	1:1.3
Volumes				
D/S Rockfill	cu yd	4,126,000	4,880,000	4,130,000
U/S Rockfill	cu yd	2,438,000	2,310,000	1,520,000
Filters	cu yd	468,000	1,100,000	1,600,000
Impervious Core	cu yd	449,000	2,140,000	1,910,000
Total Volume	cu yd	7,481,000	10,400,000	9,160,000

Significant savings on excavation and rockfill volumes were effected at Brownlee by designing the rockfill to be founded on as much as 100 ft of existing consolidated sand and gravel, rather than bedrock. As pointed out previously, settlement rates have been low, indicating that the foundation material is, for all practical purposes, as satisfactory as bedrock.

The writer wishes to express his appreciation of the interest taken by the contributors to the discussion. Active participation of these interested persons has made possible a more comprehensive presentation for inclusion in this Symposium on Rockfill Dams. It is through such activities that significant technical information may be disseminated throughout the world for the benefit of all.

An aerial view of the completed Brownlee project is shown in Fig. M1.

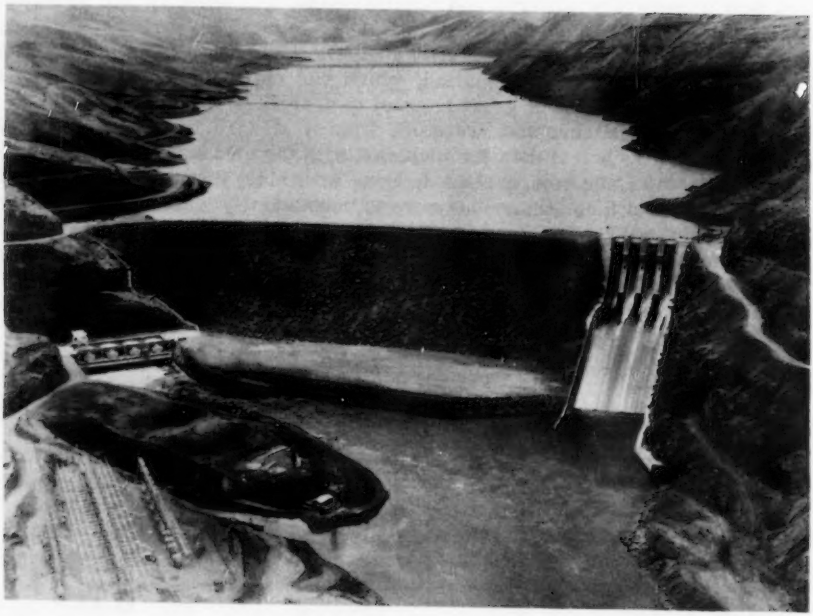

FIG. M1.—BROWNLEE HYDROELECTRIC PROJECT

HYDRAULICS OF CIRCULATING SYSTEMS[a]

Closure by Clifton W. Bolieau

CLIFTON W. BOLIEAU,[1] F. ASCE.—The interest shown by Messrs. Richards and Johnson is appreciated. Mr. Richards questions the need of two-speed operators on the pump discharge valves, and, in some cases, even the need of valves. With only one pump per condenser a valve is unnecessary, but in central-station plants two pumps per unit seems imperative to assure continuous power generation when one pump is out of service, even though plant efficiency is reduced. With two pumps per condenser, valves are quite necessary so that one pump may be out of service for repair. The small additional cost of a two-speed operator is insignificant for the additional benefits derived. A slow-opening valve will greatly reduce pressure rises in the circulating system and condenser on startup. With one of a pair of pumps being shut down, a quick-closing valve reduces the chance of backflow through the stopped pump. Mr. Johnson's discussion further confirms the desirability of two-speed valves.

The writer's reason for throttling the pumps under low-head conditions is not, as stated by Mr. Richards, to prevent pumps from operating too far out on their curves but to prevent too high condenser-tube velocities which would cause tube cavitation.

Mr. Johnson's suggestion of utilizing the head introduced in the system at low river stages for operating turbines was considered at TVA's Watts Bar and Shawnee Steam Plants. At the former plant no pumps are required; water flows by gravity from a hydro-plant reservoir just upstream from the steam plant. However, studies showed that the amount of power which could be produced under the low heads and short periods available would not pay for the structures and equipment necessary. There is no question but at other sites turbines could be justified, and each plant should be studied with this in mind.

In the matter of filling the system it must be admitted that in spite of instructions to the contrary, an operator occasionally does fill the system with the circulator pump and discharge valve throttled. However, this is dangerous because the air in the water boxes must be exhausted through a relatively small vent pipe which has a much greater capacity for air than water. When the boxes are full and the vents have to change suddenly from carrying air to water, there will be a sudden pressure rise in the boxes which may put undue strains on the tube sheets and start leaks which will contaminate the condensate. With most new plants now using once-through boilers, this fact becomes more important than ever.

In reference to Mr. Johnson's comment about the field tests of the Widows Creek Steam Plant Unit 7 pumps, these pumps are installed but cannot be tested until the condenser and turbine are more nearly complete. It is interesting

[a] February, 1959, by Clifton W. Bolieau.

[1] Prin. Mech. Engr., Div. of Design, TVA, Knoxville, Tenn.

to note that the pump manufacturer tested one of the pumps at full speed in his shop before shipment. This test was witnessed by the writer and although much of the flow was beyond the limits of calibration of the Venturi meter used, a good basis of comparison is available.

CONTROL OF CRACKING IN CONCRETE GRAVITY DAMS[a]

Discussion by Bryant Mather

BRYANT MATHER.[1]—The authors' comprehensive description of Corps of Engineers practice, its evolution and effectiveness, will be of great and continuing value to all who are interested in the problem of the minimization of temperature cracking in concrete structures. The authors, quite properly, go back to first principles in stating: "When tensile stress in excess of tensile strength develops, a crack occurs." Their comments on methods for advantageously controlling the stress level developed in the concrete, or the strength of the concrete, are, however, generally limited to the factors having direct effects on heat development and heat loss. They mention that "very little compressive stress develops as a result of the initial rise in temperature because of low modulus of elasticity and the relatively large amount of creep which occurs," but they do not follow these points up by suggesting the possibility of reducing the tensile-stress level in the concrete during cooling by intentional selection and use of materials so as to produce concrete of reduced modulus of elasticity and increased creep. It would appear that whatever practical judicious means could be taken to produce concrete with a relatively low modulus of elasticity at later ages during cooling, they would be beneficial from the standpoint of preventing cracking.

The authors refer to the necessity for "the use of well and uniformly graded aggregates," in order to achieve "uniform and sustained use of the lowest practical cement factors." They also note that the thermal properties of the concrete "are largely determined by the thermal properties of the aggregates," and that these "influence both the magnitude and rate of heat loss and temperature changes." They then state that the thermal properties are "for all intents and purposes not subject to control," since "the aggregates used in a structure are largely fixed by availability of suitable materials and economic considerations."

The purpose of this discussion is to call attention to the possible influence of factors, other than the thermal properties, that depend on the nature of the aggregates and the way that they are used in the concrete, on the crack resistance of the concrete.

In the authors' comments on the experience of the Corps of Engineers, they point to the John H. Kerr Dam as characterised by a "phenomenal" absence of cracking, and to the experience at Pine Flat and Detroit Dams as "good," noting that on these latter two the experience was "excellent" in comparison to that on Norris Dam. One characteristic that John H. Kerr, Pine Flat, and Detroit Dams have in common is that the concrete aggregate was siliceous;

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[a] October, 1959, by William R. Waugh and James A. Rhodes.

[1] Chf., Special Investigations Branch, Concrete Div., U.S. Army Engr., Waterways Experiment Sta., Jackson, Miss.

the aggregate used in the concrete in Norris Dam and in several of the other dams mentioned in the authors' paper was limestone.

Katherine Mather[2,3,4] has cited some of the results of laboratory studies of 10-in. diameter cores from five of the dams mentioned by the authors. The fine and coarse aggregate used in the concrete in three of the dams was manufactured from limestone. In one, it was manufactured from granodiorite, and in one it was manufactured from quartz diorite. Each dam was represented by cores from exterior and from interior concrete, and information was provided on the location in the structure from which the cores were taken, the age of the concrete when the cores were drilled, and the mixture proportions and aggregate gradings. Four of the aggregates had previously been tested at the U. S. Army Engineer Waterways Experiment Station, so that results of the standard physical tests, of freezing-and-thawing tests of concrete specimens, and of petrographic examinations were available. All of the cores were examined by careful inspection without the use of a microscope, the coarse aggregate content and air content of the hardened concretes were determined, sawed and broken surfaces were examined at low magnifications, and many thin sections of the concrete were prepared and studied at high magnifications.

The three limestone aggregates were not extreme variants in the limestone group, but they did differ considerably in structure, texture, and composition. One was a relatively pure calcitic limestone, on the average finer-grained and more even-grained than the other two. The second was oolitic and fossiliferous limestone grading into sandy and shaly limestone. It ranged in grain size from very fine-grained to coarsely crystalline. The third contained about 25% of chert, was the most cohesive of the three, and was moderately fine-grained. The two siliceous aggregates were medium-grained, slightly foliated granodiorite and moderately coarse-grained quartz diorite. The grandiorite was not altered but had some internal microfractures. The diorite was altered but was thoroughly interlocked by intergrowths of feldspar and quartz in the groundmass.

Comparisons of thin sections of the concrete made with the limestone aggregates and those made with siliceous aggregates revealed several conspicuous differences between them. One was the difference in the crystal size of the calcium hydroxide, which is one of the products of the hydration of portland cement, between the concretes with limestone aggregate and the concretes with siliceous aggregate. Those with limestone aggregate had relatively small calcium hydroxide crystals, and those with siliceous aggregates, relatively large ones. This general relation, observed in all of these cores, has also characterized all of the comparisons made thus far between undeteriorated concretes with limestone aggregates and comparable concretes with siliceous aggregates. It was also found between concretes of the same age made with the same portland cement and what was intended to be the same grading of the fine aggregate.

The second consistent difference between the concretes with limestone aggregate and the concretes with siliceous aggregates was the difference in the amount and particle-size distribution of material finer than the No. 200 sieve in the aggregates in the concrete. The difference in crystal size of the calcium

2 "Crushed Limestone Aggregates for Concrete," by Katharine Mather, Mining Engineering, October, 1953, pp. 1022-1028.

3 "Applications of Light Microscopy in Concrete Research," by Katharine Mather, ASTM Symposium on Light Microscopy, Special Tech. Publication No. 143, 1952, pp. 51-70.

4 "Petrographic Examination," by Katharine Mather, in section on "Hardened Concrete" in "Significance of Tests and Properties of Concrete and Concrete Aggregates," ASTM Special Tech. Publication No. 169, 1955, pp. 68-80.

hydroxide, and the difference in abundance and size of the aggregate particles finer than the No. 200 sieve, implies another point readily observed in the thin sections: There are larger areas of apparently amorphous, hydrated cement gel uninterrupted by crystalline particles in the concretes with siliceous aggregates. This observation correlated with the tendency of the cores from which the thin sections were made to develop cracks. When sawed surfaces of the concrete cores were examined at low magnification after several months exposure to laboratory air, all of the limestone-aggregate concretes had more cracks per 100 sq in. than either of the siliceous-aggregate concretes.

The consistent differences in amount and particle-size distribution of the aggregate finer than the No. 200 sieve depend on differences in physical properties of the mineral constituents of the two classes of aggregates. The percentages passing the No. 200 sieve in the fine aggregate gradings, as reported, showed an average of 3.5% in the two consisting of siliceous materials, and from 4.8% to 8.1% in the three consisting of limestone. The actual quantity of aggregate finer than the No. 200 sieve in the concrete included the reported amount, as well as an additional amount consisting both of that adhering to the aggregate particles of larger size as batched and not allowed for, and that produced by wear between the point where samples were taken for sieve analysis and the final location of the aggregate in the concrete.

In the crushing of brittle crystalline solids, the amount of new surface formed per unit of energy input increases as the hardness on Moh's scale decreases.[5] Quartz has a hardness of 7; the feldspars range from 6 to 6.5; calcite has a hardness of 3. The same expenditure of energy, therefore, yields about 1.8 times as much new surface on calcite as on quartz or feldspar.[5] If all the aggregates were similar in original grain size and interlock and were subjected to the same amount of wear due to impact and abrasion during batching and mixing, the amount of new surface produced on the limestones should be about 1.8 times that produced on the granodiorite or the quartz diorite, and the amounts of material finer than the No. 200 sieve should be proportionately different. As a matter of fact, the limestones were of finer grain; it was difficult to compare the degree of interlock between the limestones and the diorite and granodiorite.

There were two other features of these concretes that were related to the amount of aggregate finer than the No. 200 sieve in them. Like all portland-cement concrete placed by conventional methods, all these cores showed some evidence that the mortar had bled water and subsided under the larger coarse aggregate particles before set. However, the limestone-aggregate concrete bled less and subsided less under the aggregate than the siliceous-aggregate concretes. These observations may be explained as due to the effect of the presence of differing amounts of material of high specific surface in reducing bleeding, and the greater effectiveness in this respect of pulverized limestone as compared to pulverized quartz of similar specific surface.[6]

The additional effect of the larger amount and smaller average particle size of the limestone particles finer than the No. 200 sieve was a reduction in amount of entrained air in mortar, and a reduction in maximum and median size of the individual air voids. A given air content in voids of small size should give more frost resistance than the same air content in voids of larger size.[7] In

[5] "Basic Laboratory Studies in the Unit Operation of Crushing," by J. W. Axelson et al, Transactions, AIME, vol. 184, 1951, pp. 1061-1069.

[6] "The Bleeding of Portland Cement Paste, Mortar, and Concrete; Treated as a Special Case of Sedimentation," by T. C. Powers, Bull. 2, Research Lab., Portland Cement Assn., 1939.

[7] "The Air Requirements of Frost-Resistant Concrete," by T. C. Powers, Proceedings, Highway Research Bd. vol. 29, 1949, p. 184.

Fig. M1, the points designated Dams A, B, and E refer to the limestone-aggregate concretes; Dam C contained granodiorite aggregate; Dam D concrete contained quartz diorite aggregate.

The modulus of elasticity of the concrete in the cores from Dams A, B, and E, made with limestone aggregate, was higher than that of cores from Dams C and D, made with siliceous aggregates, although the difference was not very great. Cores from the John H. Kerr Dam with siliceous aggregate, where the absence of cracking is phenomenal, were compared to those from two other structures made using limestone aggregates, where moderate to extensive cracking has taken place. In this comparison, the micro structure of the concrete with limestone aggregate resembled that of the cores from Dams A, B, and E, while the microstructure of the concrete from the John H. Kerr Dam resembled that of the cores from Dams C and D. The modulus of elasticity of the concrete in the cores from the John H. Kerr Dam was about 0.4 that of the average of the concrete in the cores from the two other dams, made with limestone aggregates, with which it was compared.

It has recently been suggested[8] that the differences in effectiveness between limestone and sandstone additions in controlling expansion and cracking of Kansas sand-gravel aggregate concrete may be due in part to the opportunity for calcium hydroxide to form epitactic overgrowths on calcite but not on quartz. Jacques Farran[9] has reported data that demonstrate that such epitactic growth results in the development of stronger bond between portland-cement paste and clean polished calcite than develops between cement paste and clean polished quartz. The presence of a myriad of tiny calcite particles and their attractiveness as growth sites for calcium hydroxide, may account for the dissemination of the calcium hydroxide in small crystals in the matrix of the limestone-aggregate concretes.

Photomicrographs of thin sections taken in both plane light and with crossed nicols at 165 diameters magnification of interior mass concrete from Dams B, D, and E are available[2] as are similar photomicrographs of thin sections of exterior mass concrete from Dams A, B, C, and D;[3] the differences in microstructure and texture between those made with limestone and siliceous aggregates are striking.

These observations suggest that there may have been contributions to crack resistance of the concrete in dams such as John H. Kerr, Detroit, and Pine Flat by the fact that the aggregates used were siliceous. They also suggest the desirability of further studies of the possibility of reducing the degree of cracking caused in concrete as a result of given combinations of temperature history and restraint by selection and use of aggregates in ways that may result in making the concrete inherently more crack-resistant.

In selecting aggregates for use in large dams, it might be wise to take greater cognizance of the fact that concrete of low modulus of elasticity is more prone to creep and less liable to crack than is concrete with a high modulus. In cases where it appeared impractical to select siliceous aggregates that would produce more crack-resistant concrete, it might be beneficial to control the grading of the coarse aggregate carefully to produce the lowest void content, possibly by the use of gap gradings, so that the least amount of mortar

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8 Discussion of W. C Hansen, "Expansion and Cracking Studied in Relation to Aggregate and the Magnesia and Alkali Content of Cement," by Bryant Mather, Journal, Amer. Concrete Inst., Proceedings, vol. 55, September, 1959, pp. 1507-1511.

9 Contribution Minéralogique à l'Etude de l'Adhérence entre les Constituants Hydratés des Ciments et les Matériaux Enrobés," by Jacques Farran, Revue des Materiaux, Paris, Nos. 490-491, July-August 1956, pp. 191-209.

and paste would be required. Also in such cases it might be beneficial to restrict by specification the amount of material permitted to pass the No. 50, 100, and 200 sieves to smaller amounts than when siliceous aggregates are used. Finally it might be found desirable to require more air to be entrained in concrete mixtures using limestone aggregates than in concretes with siliceous aggregates.

In general, high strength and high modulus of elasticity go together. We like high-strength concrete, but in gravity dams we want low modulus of elasticity. Perhaps it would be expedient to employ other controls than economy in order to set an upper limit on strength and modulus of elasticity at the age when the concrete is subjected to the greatest potential tensile stress due to cooling and restraint. As experience with the use of pozzolans and other mineral admixtures increases, we may find that it is practical and feasible to produce adequately watertight, durable, low-modulus-of-elasticity, controlled-strength concrete by judicious use of such materials, improved cement specifications, improved aggregate grading limits, and actually design concrete for inherent crack-resistance of a degree that is now obtained only by happy accident.

Acknowledgements.—This discussion has been read and valuable contributions to it have been made by Katharine Mather and Thomas B. Kennedy, for which the writer is most grateful.

AMERICAN POWER-PLANT COOLING TOWERS[a]

Discussion by John H. Ruskin

JOHN H. RUSKIN,[1] M. ASCE.—The several basic design and construction considerations peculiar to cooling towers have been presented in usable fashion. Some of this information would otherwise be available only to those possessing a formidable library of manufacturers' catalogs, and it is well to have a readily available assembly of pertinent engineering aspects of so common a feature of engineering projects.

While Mr. Scarola showed in considerable detail the structural and fire precautions, however, it is unfortunate that he did not develop a few paragraphs to warn the designer of certain health hazards which could—and often do—become part of the cooling system. These hazards are a direct result of the inherent nature of a cooling tower, but they can be obviated by careful and intelligent design. The hazards to which this refers are those resulting from cross connections between the cooling tower sump or some part of the recirculated water system and the potable water system of the establishment.

By no stretch of the imagination can the water in a cooling tower sump or system be considered potable—that is, fit for human consumption—and no one would give serious consideration to a proposal for such use. The very nature of a cooling tower makes it function comparably to an air washer, and as such it accumulates between cleanings variously large concentrations of soot, dust, bird manure, radioactive fallout particles, airborne bacteria, algae, and any treatment chemicals used to prolong the life of the system or the usable period for the makeup water. Sussman and Portnoy[2] have described the effects of closed-system operation of cooling towers on total solids, pH, alkalinity, hardness, sulfate, ammonia, and other constituents of sanitary significance. Others also have contributed notably along this line. Bacteriological analysis of cooling water yields prolific colonies of coliform organisms and many pathogenic varieties of microscopic life.

The health hazards commonly arise in two ways: (1) At the time of installation, the designer may cause to be built into the piping system a potable water connection for bypassing all or some part of the recirculation system: or (2) a plumber or pipefitter may alter the originally safe design to accomplish the same or another purpose. In addition, many designers and installers are unaware of the necessity and means for protection against back-siphonage from the cooling tower sump or against pollution through gland seal or water lubricant arrangements connected to the potable water supply. The problem of the intentional or accidental cross connection is by far the most potent, because such a connection is usually made as large as possible. This problem can be

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[a] October, 1959, by J. A. Scarola.

[1] Assoc. Sanitary Engr., Dept. of Health, Detroit, Mich.

[2] "Water Composition Changes in Air-Conditioning Equipment," Journal, AWWA, 51: 953, August, 1959.

solved if the designer gives intelligent consideration to the probable necessity for doing so, and keeps in mind that water can flow two ways in any pipe.

Any mechanical system, including a cooling tower and the other components of a recirculated water system, are subject to mechanical failure or at least to shutdowns for maintenance purposes. These cannot always be scheduled to avoid interference with functioning of the equipment which depends on the cooling system. It is frequently the case that a dire emergency develops where much damage will be done if cooling water service is not restored in a matter of minutes. The author points this out but does not suggest a solution. The designer who is not cognizant of health hazards frequently provides for a bypass by connecting a potable water line directly to the pump discharge piping. Occasionally he will provide more than the usual single gate valve, but no combination of ordinary check and gate valves should ever be relied upon to isolate a recirculated water system from a potable water system.

One foolproof arrangement which has been utilized is a small receiving tank and a special standby pump to which potable water can be supplied through an air gap of twice the diameter of the supply pipe. The same tank can receive non-potable water if desired. Another layout is the swivel connection, as shown in Fig. 1, in which a certain part of the piping system may be switched from one system to the other, analogous to a switch in a railroad track layout. Like the latter, the piping swivel must be arranged so that one section of the path is utilized totally in one position or the other. Simultaneous supply must be impossible. For convenience, this may be accomplished in some cases by use of a four-port four-way valve of the rotary plug type, preferably a lubricated model as shown in Fig. 2. With the valve plug axis oriented horizontally and the bottom port kept open as a drain or vent, the discharge line to the equipment is connected to the top port. Potable water is then supplied to one side port and non-potable water to the opposite side port. The plug has two 90° passages so that the left and bottom ports are connected at the same time as the right and top ports, and vice versa. Interconnection of the two systems through the four-way valve so installed is, thus, impossible. Stop cams guide the operator in setting the valve plug in proper alignment with the ports in either position.

Because the piping on the discharge side of the four-way valve or other swivel arrangement is at least temporarily filled with non-potable water at time of switching to the potable supply, the latter must be protected against backflow as shown schematically in Fig. 3. On cooling tower systems a double check valve assembly is frequently used, although some designers specify a backflow preventer of the reduced-pressure-zone type, as described by Dempster,[3] et al. Care must be taken by color coding to avoid any type of bypass around the swivel or four-way valve arrangement.

Reliance on valves which do not effectively separate the potable from the non-potable system, either by an air gap, by a drained section of pipe (swivel), or by an intermediate zone of reduced pressure maintained via visible discharge during malfunction (backflow preventer), has caused serious trouble in several known cases and many more unreported cases. Sudden appearance of chromate-treated water at a drinking fountain, followed by shutdown by a large factory installation, accompanied by the manager's announcement of pollution of the water supply over a plant-wide inter-communication system, was the price paid by one establishment for poor design. In another case, sudden appearance of calcium-chloride brine from a chiller system at water outlets in the plant

3 "Use of Backflow Preventers for Cross Connection Control," Journal AWWA, 50: 1589, Dec. 1958.

cafeteria was disconcerting to patrons. A large office building manager permitted a tenant's plumber to create a cross-connection to its air conditioning equipment so that it could be operated "off-season" after cooling water from the towers was no longer being circulated. In a little-used city water line connected to a non-potable system, a series of five gate valves and two check valves were "closed," but the water meter still registered backwards under certain pressure conditions.

Other occurrences could be cited, but these should illustrate the principle that thoughtful design of a cooling tower and its appurtenant water recirculation system should include appropriate provision for emergency or standby supply from another source in a safe manner so it will not be necessary to create an improper arrangement resulting in health hazards. Omission of such a provision should be weighed against the possible results of such omission.

QUICK DESIGN OF AIR VENTS FOR POWER INTAKES[a]

Corrections

CORRECTIONS.—On page 20, in Fig. 3, the legend for the ordinate should read $\frac{P}{H}$.

[a] December, 1959, by G. S. Sarkaria and O. S. Hom.

UNDERGROUND POWER STATIONS IN ITALY[a]

Discussion by Carlos S. Ospina

CARLOS S. OSPINA,[1] F. ASCE.—The design of underground hydroelectric power stations involves design and construction complications that should make this type less attractive than surface plants in cases where the advantages of the underground alternative are not overwhelming.

Access Facilities.—From Table 1, it appears that access shafts are not used often in Italy. Of the 55 underground plants listed, only five have access shafts and, of these, one also has an access tunnel. It appears that the same tendency applies to underground stations in other countries. Construction requirements normally impose the necessity of an access tunnel.

In regard to the nine relatively long entrance tunnels (more than 400 m), it would be interesting to know more; grade, height, width, paving, etc.

Ventilation.—In regard to ventilation during construction and when the plant is in service it would be enlightening to have the authors describe the Italian practice.

Penstock Shafts.—In regard to penstock shafts it is interesting to note those of the 64 plants for which information is available:

18 have steel lined shafts.
23 have steel penstocks placed free in the tunnels.
20 have reinforced concrete lined shafts.
3 have prestressed concrete linings.

It would be interesting if the authors were to comment on present practice in this regard. Present practice in Sweden and Canada is toward unreinforced concrete linings and steel lined pressure shafts.

Are full-scale pressure tests in the rock performed in Italy prior to design?

The Sumplago penstock arrangement (three 2.55 m steel penstocks set free in one concrete-lined vertical shaft and two 2.55 m penstocks in the other shaft) does not appear at first glance as the best solution as to first cost, maintenance cost, and weight of steel. It would be of interest for the authors to provide a description of this feature and to give the criteria that led to the adoption of such an unusual design. In addition to cost there are questions like support of pipes, access for maintenance, larger shafts required, greater erection and welding costs, etc.

Bibliography.—A bibliography limited to recent projects would be of great interest.

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[a] December, 1959, by Dante Finzi, Mario Mainardis and Carlo Semenza.

[1] Partner, OLAP - INGENIERIA, Bogota, Colombia.

These conclusions and other discussions were given generous editorial emphasis in a British engineering publication.[3]

An official United States government publication[4] of special significance with respect to power development in Brazil states:

> "In 1952 construction was also started of the thermal electric plant at Piratininga, with two units generating 80,000 kilowatts each and which should start operating at the end of 1954 or beginning of 1955. New plans for the amplification of the system are not now being considered. However, at one time the possibility was considered of increasing the capacity of the thermal plant to 600,000 kilowatts. The Joint Commission very advisedly counselled that such amplification (in the power system) should not exceed two units, due to the tremendous consumption of imported oil that would seriously affect the already unfavorable commercial balance of Brazil."

Furthermore (quoting the writer):[5]

> "Where the importation of fuel demands the diversion of precious foreign exchange, which is already in short supply, the financial position of a country could by this means become very adversely affected over a period of years.
>
> "The protection of the national interest in such matters calls for far-sighted planning on the part of all who have a share in this responsibility. For a hydroelectric project, the hydrology, land acquisition and governmental concessions generally require the preliminary work to begin two to three years earlier than is needed for a thermal plant. Where such preliminaries have been neglected, the resulting power shortage may create a public clamor for the quickest possible solution, and for this a thermal plant presents a tempting answer, since its harmful effect on the country's credit position may then be claimed, in some quarters, to be a less serious prospect than the consequences of a local power shortage. The heart of the problem, therefore, lies in the wisdom and perseverance which is exercised in the preliminary planning stage of a hydroelectric project."

Unfortunately, very few North American engineers have had an opportunity to acquire an adequate understanding of what this means in the case of foreign developments. In view of current work and new proposals to build more steam generating plants, some further exposition on this subject may be generally helpful.

Since Brazil has no developed oil supply of its own, the country must, somehow, earn dollars or other foreign exchange with which to pay for imported oil. The principal means of earning foreign exchange is through the export and sale of coffee. If a proposal is advanced to construct a new steam plant which will annually burn $10,000,000 worth of oil, it automatically becomes necessary for Brazilian farmers to raise some 25,000,000 lb of additional coffee each year and to find a foreign market for it. However, this apparently simple formula, has an important defect: news reports indicate that Brazil has been plagued in

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[3] Water Power, England, March, 1955.

[4] "Brazilian Technical Studies," Joint Brazil-United States Economic Development Comm., December 31, 1953, p. 293.

[5] "Hydro—The Answer to Brazil's Power Needs," by Adolph J. Ackerman, Civil Engineering, Vol. 25, May, 1955.

PIRATININGA STEAM-ELECTRIC GENERATING STATION[a]

Discussion by Adolph J. Ackerman

ADOLPH J. ACKERMAN,[1] F. ASCE.—An engineering discussion of power development in another country implies a responsibility of relating such discussion to the economic welfare of that country. A recent report[2] stated that "the profession of civil engineering, more than any other branch of engineering, deals with many diverse problems of society," and that "the harmonizing of all the conflicting interests involved in any great civil engineering project requires a broad understanding of the political, legal, business and human elements inherent in the situation."

In this technological age, electric power is the foundation for the economic development of a country. This, of course, calls for the development of new power supplies based on the local economic conditions and under productive financial policies. Failure to fully understand the significance of this principle and its proper applications can readily contribute to the gradual destruction of a country's economy. This problem was examined at the World Power Conference in Rio de Janeiro in August, 1954, where the writer presented the following view in a paper entitled "Planning of the Electric Power Industry in Brazil":

> "Brazil is blessed with great resources of hydroelectric power. The development of these resources is fundamental to the country's future economic growth and industrialization. In this respect the history of power development in Canada will probably be the best pattern. At the end of 1952, Canada had an installed generating capacity of 14,300,000 h.p. in hydroelectric power. Of the total energy production, 96.5 per cent was hydroelectric power, and only 3.5 per cent was thermal power, because Canada's fuel resources had not (until very recently) been developed.
>
> "As long as Brazil's oil and coal resources remain undeveloped, and as long as foreign exchange is needed for the importation of essentials other than such fuels, Brazil is confronted with the necessity of developing its hydroelectric resources to the fullest extent possible. A simple formula to keep in mind is the following: The annual payments for imported fuel oil consumed in a steam plant (operating at a high load factor) would be equal to the annual charges on a loan to finance the importation of all equipment for a hydroelectric development whose generating capacity would be 6 to 8 times greater than the capacity of the steam plant."

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[a] April, 1960, by O. L. Hooper and H. M. Estes.
[1] Cons. Engr., Madison, Wis.
[2] ASCE Task Committee On Professional Educ., 1958.

recent years with great surpluses of coffee for which there is no market. The alternative then is to divert $10,000,000 every year from the limited available foreign exchange which could otherwise be used for the importation of machine tools and similar productive essentials.

It is distressing, to say the least, to observe the current breakdown of that country's economy. Any effort to rectify this trend must, of necessity, be founded on a sound program of developing the country's power supply. It is incumbent upon North American engineers to fully understand what this means if they commit themselves to working in Brazil, or in other countries with similar problems.

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

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c. Discussion of several papers, grouped by divisions.

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